FEBRUARY 1958

Journal

AMERICAN WATER WORKS ASSOCIATION

VOL. 50 • NO. 2

Heavy, heavy . . . construction, that is, as Charlotte, N.C., grows, grows

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Dye, Hudson

NEBRASKA WATER RESOURCES LAW
Perry

ASPECTS OF SAFETY
Rener, Housel

STANDARD FOR LIQUID CHLORINE
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February 1958

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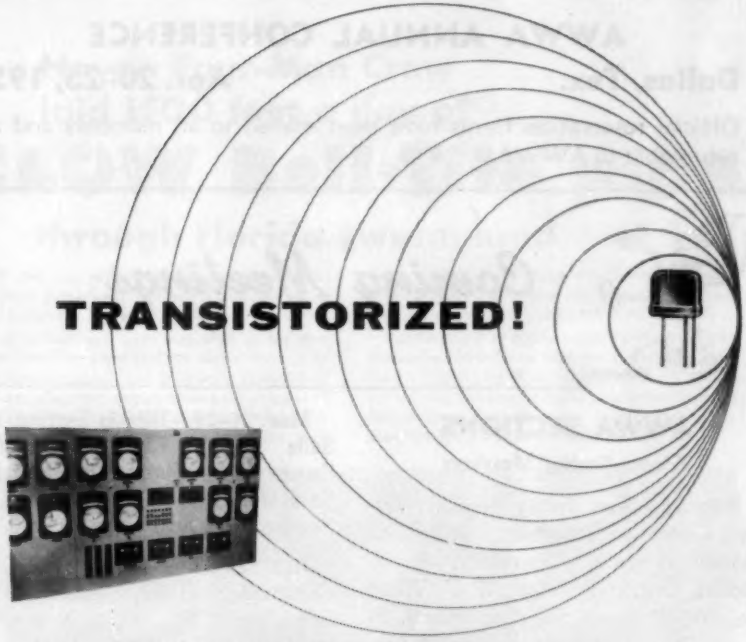

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Official reservation forms have been mailed to all members and are returnable to AWWA.

Coming Meetings

AWWA SECTIONS

Winter-Spring Meetings

Feb. 13—New Jersey Section Winter Luncheon Meeting, at Essex House, Newark. Secretary, A. F. Pleibel, Dist. Sales Mgr., R. D. Wood Co., 683 Prospect St., Maplewood.

Mar. 12–14—Kansas Section, at Lamer Hotel, Salina. Secretary, Harry W. Badley, Representative, Neptune Meter Co., 119 W. Cloud, Salina.

Mar. 20–22—Montana Section, at Florence Hotel, Missoula. Secretary, Arthur W. Clarkson, Asst. Director, Div. of Environmental Sanitation, State Board of Health, Helena.

Mar. 23–25—Southeastern Section, at Dinkler-Plaza Hotel, Atlanta, Ga. Secretary, N. M. deJarnette, Engr., Div. of Water Pollution Control, State Dept. of Health, 309 State Office Bldg., Atlanta, Ga.

Mar. 26–28—New York Section, at Van Curler Hotel, Schenectady. Secretary, Kimball Blanchard, Riversville Rd., Greenwich, Conn.

Mar. 26–28—Illinois Section, at LaSalle Hotel, Chicago. Secretary, Dewey W. Johnson, Research Engr., Cast Iron Pipe Research Assn., 3440 Prudential Plaza, Chicago.

Apr. 16–18—Nebraska Section, at Cornhusker Hotel, Lincoln. Secretary, Rupert C. Ott Jr., Neptune Meter Co., 2818—21st St., Columbus.

May 15–17—Pacific Northwest Section, at Davenport Hotel, Spokane, Wash. Secretary, Fred D. Jones, Asst. Supt., Water Dept., 306 City Hall, Spokane, Wash.

May 15–17—Arizona Section, at El Conquistador Hotel, Tucson. Secretary, Stanford I. Roth, Supervisor of Water Collections, Div. of Water & Sewers, Phoenix.

Jun. 1–4—Canadian Section, at Royal York Hotel, Toronto, Ont. Secretary, A. E. Berry, Gen. Mgr. & Chief Engr., Ontario Water Resources Commission, Parliament Buildings, Toronto.

Jun. 25–27—Pennsylvania Section, at Hotel Lawrence, Erie. Secretary, L. S. Morgan, Div. Engr., State Dept. of Health, Greensburg.

(Continued on page 8)

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Coming Meetings

(Continued from page 6)

Fall Meetings

Sep. 8–10—Michigan Sec., Grand Rapids.

Sep. 10–12—New York Sec., Lake Placid.

Sep. 17–19—Ohio Sec., Cleveland.

Sep. 17–19—Wisconsin Sec., Wausau.

Sep. 22–24—Kentucky-Tennessee Sec., Memphis, Tenn.

Sep. 24–26—North Central Sec., Duluth, Minn.

Sep. 28–30—Missouri Sec., Jefferson City.

Oct. 15–17—Iowa Sec., Des Moines.

Oct. 23–24—West Virginia Sec., Charleston.

Oct. 23–25—New Jersey Sec., Atlantic City.

Oct. 28–31—California Sec., Los Angeles.

Oct. 29–31—Chesapeake Sec., Wilmington, Del.

Nov. 5–7—Virginia Sec., Richmond.

Nov. 10–12—North Carolina Sec., Greensboro.

OTHER ORGANIZATIONS

Feb.–Apr.—Short courses sponsored by R. A. Taft Sanitary Engineering Center: Detection and Control of Radioactive Pollutants in Water (Feb. 24–28), Advanced Training for Sanitary Engineers in Water Supply and Water Pollution (Mar. 3–14), Sanitary Engineering Aspects of Nuclear Energy (Mar. 17–28), Basic Radiological Health (Apr. 28–May 9). Apply to: Chief, Training, R. A. Taft Sanitary Engineering Center, 4676 Columbia Pkwy., Cincinnati 26, Ohio.

Feb. 13–15—National Society of Professional Engineers, Michigan State Univ., East Lansing, Mich.

Feb. 24–28—American Society of Civil Engineers, Hotel Sherman, Chicago, Ill.

Mar. 2–7—Texas Water & Sewage Works Assn. Short Course, Texas A&M College, College Station, Tex.

Mar. 17–18—Steel Founders' Society of America, Drake Hotel, Chicago, Ill.

Mar. 17–21—4th EJC Nuclear Engineering & Science Conference and International Atomic Exposition, International Amphitheater, Chicago, Ill.

May 5–7—Purdue Univ. Industrial Waste Conference, Purdue Memorial Union Bldg., Lafayette, Ind.

May 12–14—Analysis Instrumentation Div., Instrument Society of America, Shamrock Hilton Hotel, Houston, Tex. For registration information write: H. S. Kindler, ISA Director of Technical Programs, 313—6th Ave., Pittsburgh, Pa.

May 30–Jun. 1—European Organization for Research on Fluorine and Dental Caries Prevention, Brussels, Belgium (in connection with World's Fair). For information write: Dr. M. Joachim, Chairman, Organizing Committee of ORCA Congress, 67 Rue de Treves, Brussels, Belgium.

Jun. 10–12—Appalachian Underground Corrosion Short Course, School of Mines, West Virginia Univ., Morgantown, W.Va.

Jun. 22–27—American Society for Testing Materials, Hotel Statler, Boston, Mass.

Sep. 1–13—2nd International Conference on Peaceful Uses of Atomic Energy, Geneva, Switzerland.

Oct. 5–9—Federation of Sewage & Industrial Wastes Assns., Detroit, Mich.

Oct. 13–17—American Society of Civil Engineers, New York, N.Y.

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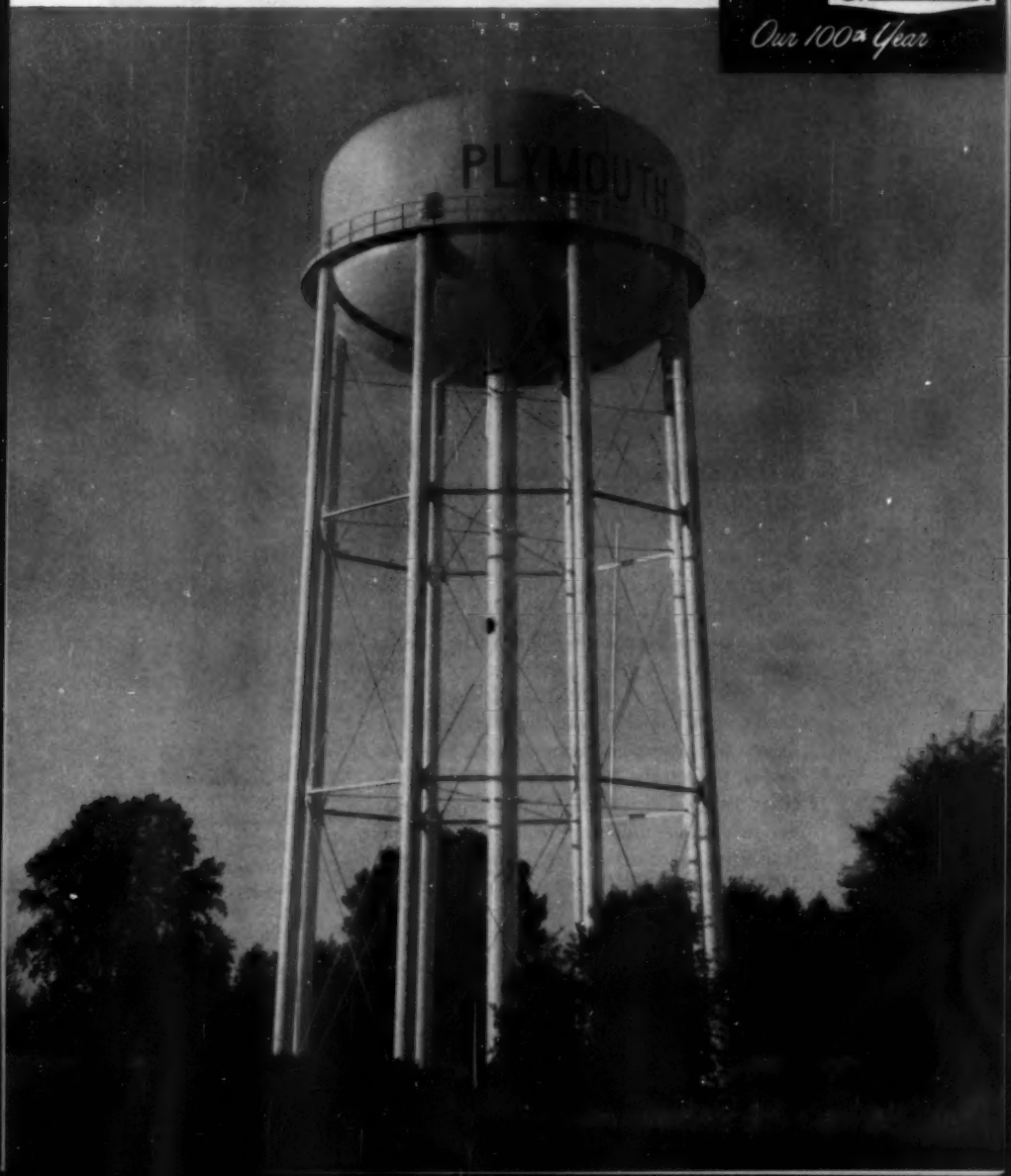

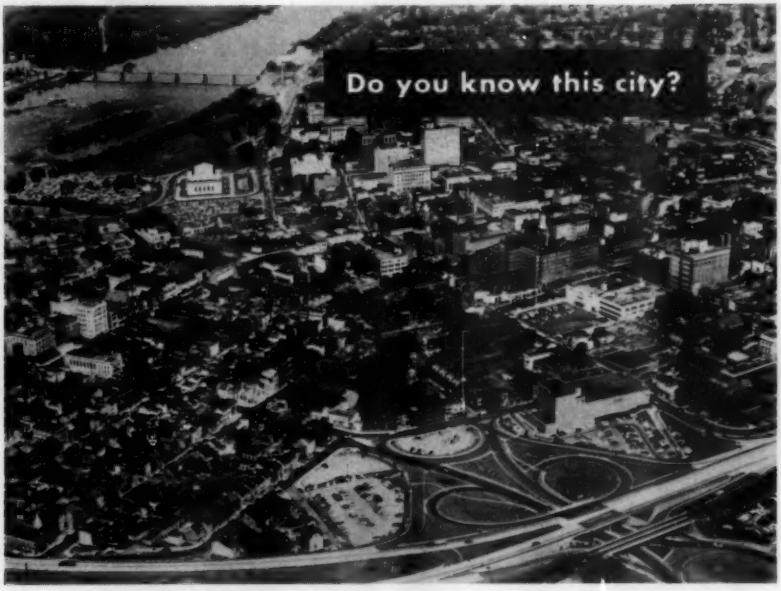

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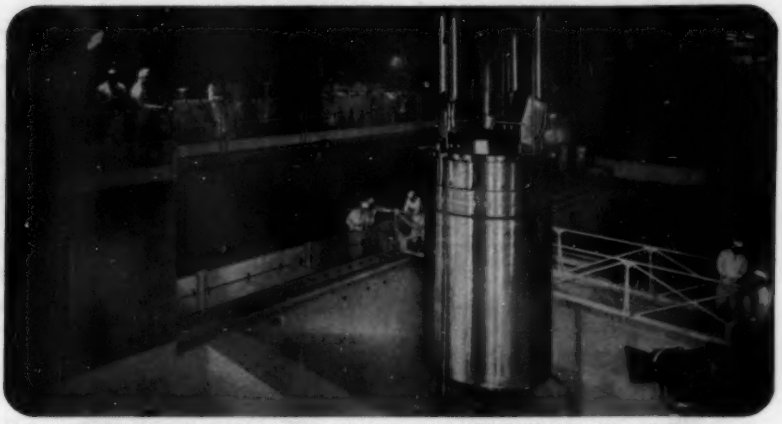

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At the SHIPPINGPORT ATOMIC POWER STATION—where water conditioning must be efficient . . . dependable . . . fool-proof—AMBERLITE Ion Exchange Resins are used.

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Whatever your water conditioning problem, your engineering company specializing in water conditioning is qualified by experience to recommend how AMBERLITE resins can best serve your special needs. For detailed information about AMBERLITE resins, write for our booklet *"If You Use Water"*.

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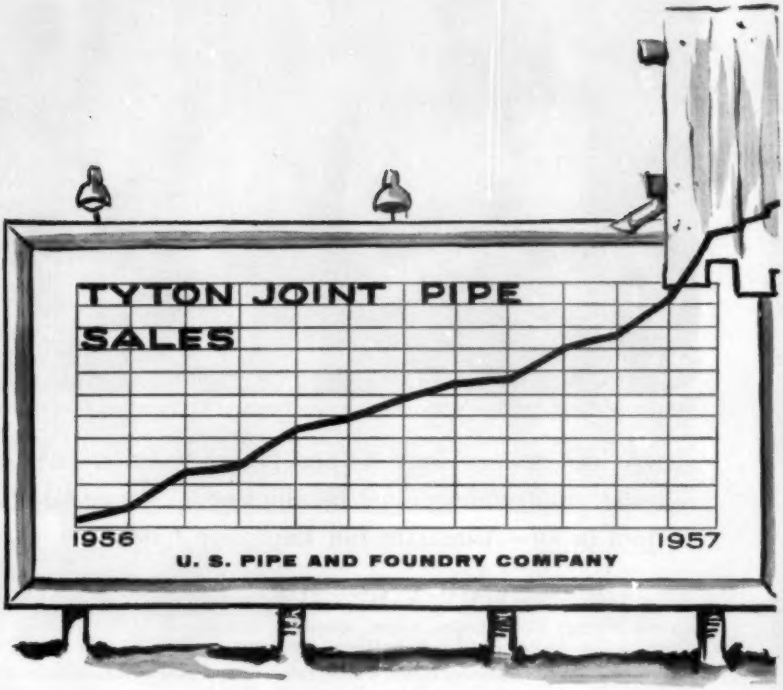
ROHM & HAAS COMPANY

THE RESINOUS PRODUCTS DIVISION

Washington Square, Philadelphia 5, Pa.

Representatives in principal foreign countries



makes friends

Tyton Joint® pipe was introduced a little over a year ago. Since then its acceptance and performance have been phenomenal.

And no wonder. Tyton Joint delivers everything a pipe user needs... and then some. It's easy to assemble—even a green crew gets the hang of it in no time.

Only one accessory needed... a specially designed rubber gasket that seats into the bell. When the entering pipe slides in, the gasket is compressed to give you a

TYTON U.S. PIPE FOR WATER, SEWERAGE AND

everywhere

permanent, bottle-tight seal. No bell holes. No caulking equipment needed. No nuts or bolts to fasten. No worry about weather, either. You can lay Tyton Joint pipe in rain or wet trench.

Like to know more about "Tyton," the ingenious pipe joint that cuts costs, saves time, sidesteps trouble? Call or write today.

U. S. PIPE AND FOUNDRY COMPANY
General Office: Birmingham 2, Alabama

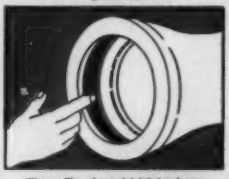
A WHOLLY INTEGRATED PRODUCER FROM MINES AND BLAST FURNACES TO FINISHED PIPE

INDUSTRIAL SERVICE CAST IRON

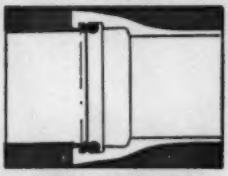
TYTON

ONLY FOUR SIMPLE ACTIONS

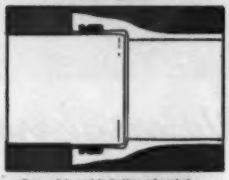
Insert gasket with groove over bead in gasket seat



Wipe a film of special lubricant over inside of gasket



Insert plain end of pipe until it contacts gasket



Force plain end to bottom of socket . . . the job's done!

28" Tyton Joint pipe being installed for water supply line in Alabama.

Waterspheroid at Ocala, Florida

This 500,000-gallon Waterspheroid stores 500,000 gallons of water and serves the Northeast section of the City of Ocala. The tank, used for general service and fire protection, has a diameter of 55 feet, 6 inches. The tank is 132 feet, 6 inches to the bottom capacity line.

Horton Ellipsoidal bottom elevated tanks, standpipes, reservoirs, radial-cone tanks and Waterspheres are also available to meet your water storage requirements. Write our nearest office for an estimate or information.

CHICAGO BRIDGE & IRON COMPANY

Plants in Birmingham, Chicago, New Castle, Del., Salt Lake City and Greenville, Pa.

ATLANTA	CLEVELAND	NEW YORK	SAN FRANCISCO
BIRMINGHAM	DETROIT	PHILADELPHIA	SEATTLE
BOSTON	HOUSTON	PITTSBURGH	SOUTH PASADENA
CHICAGO	NEW ORLEANS	SALT LAKE CITY	TULSA

In Canada—HORTON STEEL WORKS, LIMITED, FORT ERIE, ONT.

Want cobwebs on *your* complaint phone?

Then use Calgon* — for Calgon stops red water and lime scale — the two main causes of customer complaints. Yes, you can turn complaints into compliments, because whether the "red water" comes from iron present at the source, or iron dissolved from the pipes by corrosion, Calgon treatment is the answer.

When well waters containing iron and manganese are treated with a few parts per million of Calgon, the iron and manganese are kept in solution and the water is as clear at the consumer's tap as it was when pumped from the well.

When red water is caused by iron dissolved from the pipes, it can be prevented with Calgon† treatment because Calgon forms a protective film on metal surfaces. Moreover, this protective film also reduces costly tuberculation damage, flows are kept high and pumping costs are reduced. Calgon treatment is particularly effective after water main cleaning since the protective film forms quickly and tuberculation never gets a chance to start.

A report of a twenty-four month study on the efficiency of Calgon in maintaining pipeline flows is available. Ask for a reprint of this paper.

Write or phone for more information. An experienced Calgon engineer will be glad to make detailed recommendations on your specific problem.

CALGON COMPANY

DIVISION OF **HAGAN** CHEMICALS & CONTROLS, INC.
HAGAN BUILDING, PITTSBURGH 30, PENNSYLVANIA
DIVISIONS: CALGON COMPANY, HALL LABORATORIES
IN CANADA: HAGAN CORPORATION (CANADA) LIMITED, TORONTO

* Calgon is the Registered Trade Mark of the Calgon Company for its glassy phosphate (sodium hexametaphosphate) products.
† Fully licensed for use under U. S. Patents 2,337,856, 2,304,850 and 2,358,222.

This pipe may have to carry

1978 WATER

at 1958 rates.....

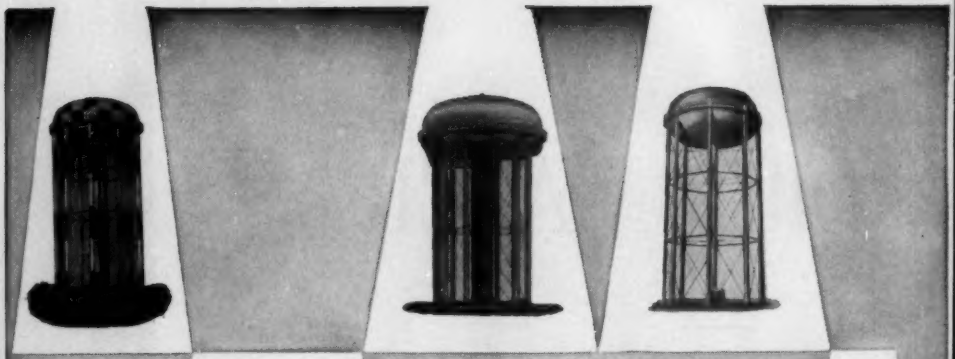
Even today after two decades of rapidly advancing costs, many cities have been forced to operate without rate increases for periods of twenty years or more. What the next twenty years may bring in operating costs no one, of course, knows. It is certain, though, that the water carrying installations using concrete pressure pipe will be in the best position to keep costs at the very minimum.

Concrete pressure pipe has virtually complete freedom from corrosion, and tuberculation. This means not only minimum maintenance costs but also means sustained carrying capacity to keep pumping costs low. Concrete pressure pipe also has the elasticity to resist bursting from surge and water hammer. And no other pipe is easier to install.

Concrete Pressure Pipe

WATER FOR GENERATIONS TO COME

AMERICAN CONCRETE PRESSURE PIPE ASSOCIATION
228 North LaSalle Street
Chicago 1, Illinois



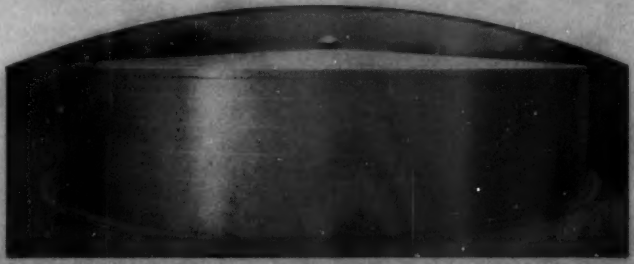
HAMMOND

safe . . . reliable

WATER STORAGE

any size—anywhere

Hammond Steel Standpipes, Steel Reservoirs, Elevated Water Tanks, Water Spheres, Filtration and Purification Units for General Water Supply & Fire Protection Systems . . . are built to all standard codes and specifications including those of the American Water Works Association, Associated Factory Mutual Fire Insurance Companies, the National Board of Fire Underwriters, the Factory Insurance Association.

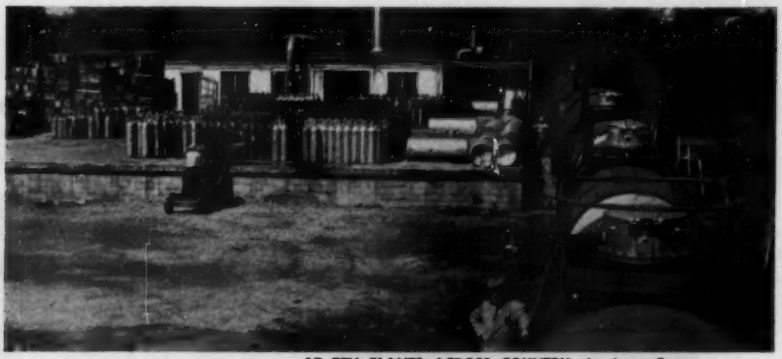


Catalog H_2O details and describes all HAMMOND Water Storage and Processing Vessels . . . Write for your copy

HAMMOND IRON WORKS

WARREN and BRISTOL, PA. • PROVO, UTAH
CASPER, WYOMING • BIRMINGHAM, ALABAMA

Sales offices throughout the U. S. A.; licensees and sales offices in many foreign countries including Argentina, Belgium, Brazil, Canada, Columbia, Egypt, England, France, Haiti, Italy, Japan, Mexico, Netherlands, Peru and West Germany.



AT TEN PLANTS ACROSS COUNTRY, the Jones Company receives tank cars of Chlorine, repackages it in cylinders and ton-tanks, makes quick deliveries to users in area.

CHLORINE

Fast delivery on less-than-carload lots

Pick Your Own Chlorine Product and Container
LIQUID CHLORINE
In 16-, 105-, 150-lb. cylinders and 1-ton tanks.
CALCIUM HYPOCHLORITE
(70% available Chlorine) 3¼- and 5-lb. tins, 100-lb. drums.
SODIUM HYPOCHLORITE
Gallons, carboys. Tankwagon deliveries in 1000-3000 gal. lots.

You get swift delivery—and NO storage worries—when you turn your Chlorine problem over to John Wiley Jones Company. Ten plants, strategically located across the U.S. stand ready to supply you.

Order shipments as small as a 16-lb. cylinder of liquid Chlorine, and as large as you want. With this flexibility, you eliminate storage problems and have complete protection against sudden emergency needs for Chlorine.

Jones Chlorine is quality controlled to meet high government standards. Our trained technical staff will be glad to help you solve your Chlorine problem. Write for prices.

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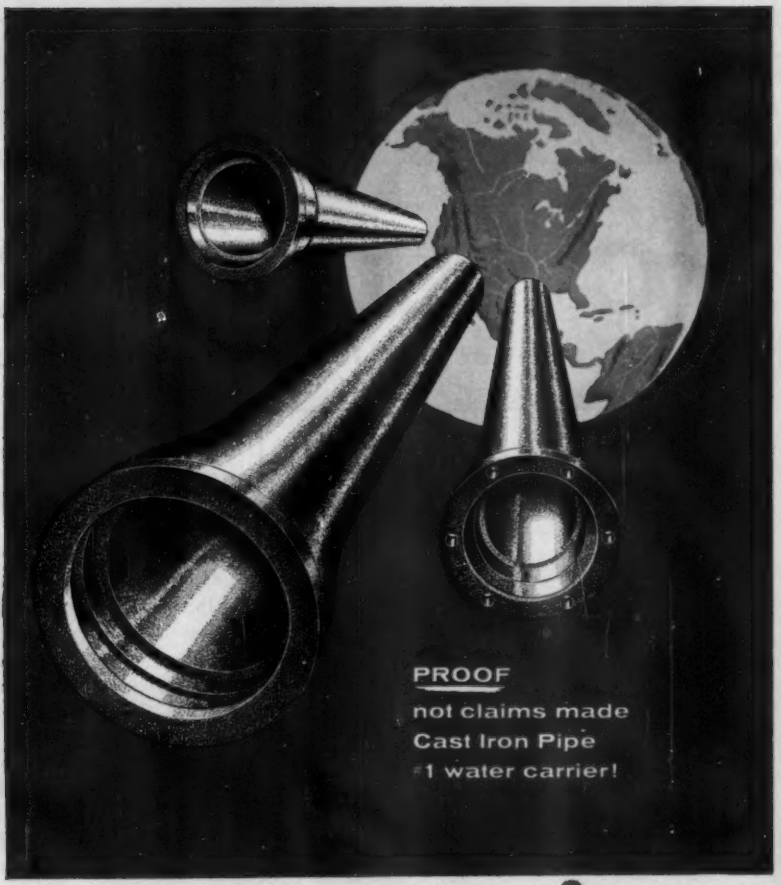
TAKE THE DOUBT



MODERNIZED **cast iron**

OUT OF PIPE BUYING

Specify Modernized Cast Iron Pipe and be SURE!

6 reasons why Cast Iron Pipe is #1 choice of U.S.A.

1. **EXTERNAL LOAD RESISTANCE...**
   6" Class 150 Pipe withstands a crushing load of 17,900 pounds per foot ... nearly 9 tons.
2. **HIGH FLOW CAPACITY...**
   Cement lined cast iron pipe and fittings will not tuberculate ... delivers a full flow for the life of the pipe.
3. **LONG LIFE...**
   42 North American cities are still using cast iron water mains laid 100 years and more ago. Hundreds more have passed the 50 year mark.
4. **BEAM STRENGTH...**
   Cast Iron Pipe is inherently tough ... stands up under heavy traffic load, soil displacement and disturbance.
5. **CORROSION RESISTANCE...**
   Cast Iron Pipe effectively resists corrosion ... vital factor in its long life and dependability.
6. **TIGHT JOINTS...**
   A full range of leak-proof, low cost, easy-to-assemble joints for pipe and fittings are available for all conditions.

Before you choose any pipe remember this important fact ... it will save you many a headache later.

In all the history of water systems no substitute pipe has ever come close to matching cast iron pipe's century old record of dependability, long life and long term economy.

Specify cast iron pipe, and your pipe worries are over, not just beginning. Cast iron's performance is based on PROOF not claims!

THE MAN WHO CHOOSES CAST IRON PIPE TODAY WON'T PAY FOR IT AGAIN TOMORROW!

Cast Iron Pipe Research Association
Thos. F. Wolfe, Managing Director
Suite 2440, Prudential Plaza, Chicago 1, Ill.

pipe FOR MODERN WATER WORKS

Engineering PLUS...

Keeps Tank Interiors Corrosion Free at a Practical Cost

One of the five municipal tanks serving the City of Jacksonville, Fla. which has been cathodically protected by E.R.P. system since 1942.

The City of Jacksonville, Fla. has relied on engineering plus from Electro Rust-Proofing Corp. since 1942 to protect water storage tanks serving the city. Since installation of cathodic protection, tanks have been virtually corrosion free on the under water surfaces and maintenance costs have been reduced to a minimum.

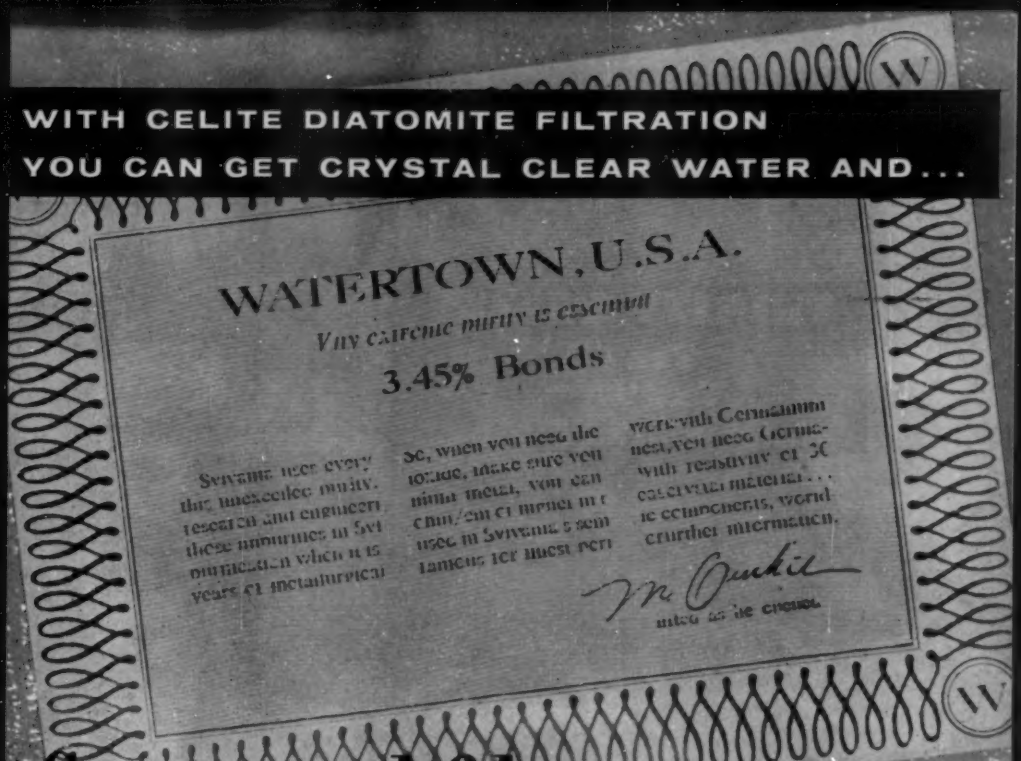
E.R.P. engineering plus means cathodic protection systems
designed to meet your needs...
maintained to your benefit...
priced to your profit...

E.R.P. provides engineering plus to meet any corrosion problem of submerged or buried metals. For full information on cathodic protection for your water storage tanks, water treatment structures and pipelines, write for bulletin E-42.

E.R.P. CATHODIC PROTECTION SINCE 1935

ELECTRO RUST-PROOFING CORP.
A SUBSIDIARY OF WALLACE & TIERNAN INC.
30 MAIN STREET, BELLEVILLE 9, NEW JERSEY
CABLE: ELECTRO, NEWARK, N. J.

WITH CELITE DIATOMITE FILTRATION
YOU CAN GET CRYSTAL CLEAR WATER AND...



Cut capital filtration costs as much as 45%

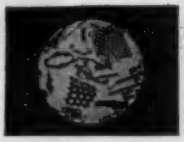
An actual installation by Johns-Manville established that a Celite* filter station can be constructed for 55% of the cost of a comparable sand filter plant.†

Diatomite systems not only cut costs, but, under comparable conditions, they also improve water clarity. For with Celite, turbidity is usually lower since more suspended impurities, including all floc, amoebae and algae, are removed. In fact in some cases, turbidity is so low it can't be measured.

Diatomite systems save space, too. To deliver the same installed capacity requires a sand filter plant 4 times as big.

Mined by Johns-Manville at the world's largest and purest commercial diatomite deposit, Celite is carefully processed for purity and uniformity. It is available in a wide range of grades to deliver the best practical balance of clarity and flow rate with any suitable filter. For further information see your nearby Celite engineer or write for free technical reprints and illustrated brochure to Johns-Manville, Box 14, New York 16, N. Y. In Canada, Port Credit, Ontario.

Celite filter aids are composed of microscopic, irregularly-shaped particles like these. 90% of a given quantity of Celite is composed of countless channels and voids that trap the finest impurities while permitting the free passage of clear liquid.



*Celite is Johns-Manville's registered trade mark for its diatomaceous silica products. †See *Comparison Studies of Diatomite and Sand Filtration* by G. R. Bell, Journal American Water Works Association, September, 1950, or write for free reprint.

Johns-Manville CELITE Filter Aids

TRINITY VALLEY

For All Cast Iron Water Works Fittings

AWWA Standard Bell Spigot Watermain Fittings—2" through 36".

Ring Tite Fittings 3" through 16" Class 150. Class 100 through 12".

Short Body and Mechanical Joint Watermain Fittings—2" through 20".

Fluid-Tite Fittings 3" through 12" Class 150.

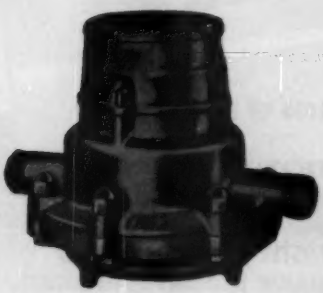
TRINITY VALLEY IRON AND STEEL COMPANY

Phone PE 8-1925 Fort Worth, Texas P. O. Box 664

The Safest Meter to Buy is AMERICAN

Because We Have Freezing Weather

"American Frost Bottom Meters have complete frost protection—frost-separable gear train and chamber, with pre-tested breakable bottom. That reduces parts replacement costs, to the absolute minimum. I repair 'em. I know American is the safest meter to buy."



• In American Meters you know in advance what will happen in freezing. The frost bottom breaks, relieving pressure, so that internal working parts are not damaged. The bottom can be replaced in a few minutes. No complicated repair jobs. No expensive working parts to replace. No extra load on the repair department. That's why Repairmen say, "American is the safest to buy!"

BUFFALO METER CO. 2916 MAIN STREET BUFFALO 14, N. Y.

Impact Insurance Against

1—LOSS OF FIRE PROTECTION.

2—EXCAVATION AND PAVEMENT REPLACEMENT.

3—COSTLY REPAIRS INVOLVING MAJOR HYDRANT COMPONENTS.

4—FLOODING.

The Smith Protectop Hydrant is designed to permit rapid return to service at minimum cost when a Hydrant is damaged as a result of a traffic accident.

The Protectop Hydrant Standpipe and Valve Stem are equipped with *Special Couplings* located just above the ground. The Couplings withstand operating pressures and ordinary impact with an ample factor of safety. Under excessive impact occasioned by traffic accidents the Couplings fracture at the design points thus minimizing the damage and permitting speedy return to service at low cost.

All Smith Hydrants are equipped with *Compression Type* Valves which definitely eliminate flooding since the line pressure holds the Valve against its seat in the closed position.

Write for details.

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THE A.P. SMITH MFG. CO.

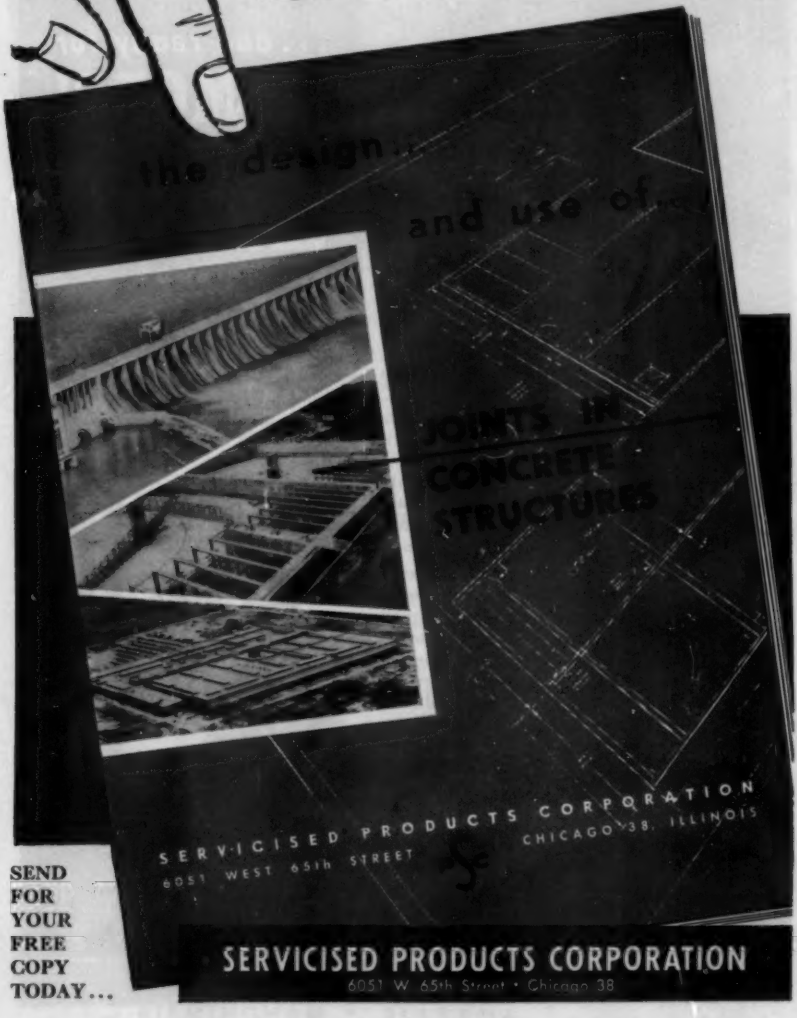
EAST ORANGE, NEW JERSEY

HERE'S COMPLETE, AUTHORITATIVE and PRACTICAL INFORMATION on

*** Design * Construction * Maintenance**

of Expansion, Contraction and Construction Joints in Concrete Structures!

Write for your free copy of this newly published manual on concrete joint design and construction. Here's complete information at your fingertips—the whole story on the design and use of joints in concrete structures. There's no obligation.



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62 YEARS' PERFECT SERVICE

... and ready for

62 MORE!

EDDY

VALVES AND HYDRANTS

Photo courtesy of Mr. W. K. Sanders, Superintendent, Water and Light Department, Morrisville, Vermont

Morrisville, Vermont, installed the above EDDY Valve in a water main way back in 1895. When uncovered recently, it was working perfectly, needing only an easily obtained, easily installed packing to restore it to 100% efficiency.

On the basis of Eddy's long time record of continuous availability of replacement parts—and in the light of innumerable cases where EDDY valves and hydrants have served perfectly, needing no maintenance, for 20, 40, 60 years and longer—we feel justified in predicting *another* 62 years' service for the valve shown here.

Interested in such service for *your* system? Then we invite your inquiry TODAY!

Eddy Hydrants and Valves meet all AWWA standards and are available to fit any existing or planned installation.

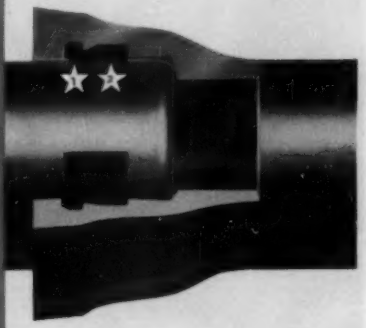
Send today for full data on the complete line of Eddy Valves, Hydrants and other waterworks' necessities. No obligation.

EDDY VALVE COMPANY

A Subsidiary of James B. Clow & Sons Inc.

WATERFORD, NEW YORK

NEW!
AMERICAN Fastite** ® JOINT* CAST IRON PIPE



The superior double-sealing☆☆ single gasket type joint

Exclusive dual-density rubber gasket provides a long-life, leak-free cast iron pipe installation...

1. **Hard rubber end provides a strong shoulder for self-centering and self-locking — a permanent seal against cold flow under pressure, fully protected against weathering and deterioration.**
2. **Softer large end for positive compression, liquid-tight joint sealing and easy assembly.**

Tests prove either hard or soft rubber section alone provides a positive seal.

The American Fastite Joint* is an all-weather-laying joint. The exclusive gasket — available *only* with Fastite pipe — is self-centering, self-locking and leak-proof. Increased internal pressures actually increase joint tightness whether pipe is laid in straight alignment or deflected.

Every feature of new American Fastite Joint* Cast Iron Pipe has been scientifically designed, laboratory and field tested, and manufactured to exact specifications with the objective of combining speed and ease of assembly with permanent, leak-free joints. Feature for feature, compare the new American Fastite Joint* with any other pipe joint now available. An American Cast Iron Pipe Company representative will be glad to tell you about the unique advantages to be had only in American Fastite Joint* Cast Iron Pipe.

FREE BOOKLET
Write for your free copy of a new booklet describing American Fastite Joint* Cast Iron Pipe. No obligation.

*Patent applied for—Underwriters' Laboratories, Inc., approved.

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AMERICAN CAST IRON PIPE CO.
BIRMINGHAM 2, ALABAMA

where ALOXITE underdrains mean economical filtration

When Cudahy put up its modern plant an important decision was made which assured both initial and long-term economy. It was this: Instead of graded gravel filter beds, Cudahy chose to support its sand filters with ALOXITE® aluminum oxide porous plates.

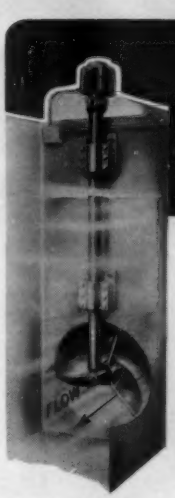
ALOXITE plates eliminated the need and cost of graded gravel and other types of underdrainage systems. And, most important, they virtually eliminated the need for filter repairs, so that the savings will increase year after year. Like all ALOXITE plate underdrains they can handle growing loads with freedom from mudballs and with minimum loss of head. And complete backwashing is accomplished without upset beds.

Construction of the new filtration plant at Cudahy was under the supervision of James Tiry, Director of Public Works and John Martinek, Water Superintendent. Answer your questions about porous media by writing for informative 56-page booklet. It's yours for the asking from:

CARBORUNDUM
Registered Trade Mark

Dept. 0-28, Refractories Division, Perth Amboy, N. J.

When measuring through the discharge end of a submerged throat or conduit, DEPEND ON . . .



10″ to 72″

Sparling
WATER WORKS INTAKE METERS

Sparling Water Works Intake Meters are designed to provide efficient and economical service. Ideal for a large volume of water flowing through the discharge end of a submerged throat or conduit. The very simplicity and variety of models available permit wide application . . . often using the existing headwalls or supports. Propellers may be plastic or stainless depending upon installation and use. Maintenance is simple. Available with Sparling's new Indicator-Totalizer 245. Drop pipes can be any length. Ideal for use in telemetering systems. 10″ to 72″

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☐ Yes! We are interested in increasing our metering efficiency. Send us more information . . . no obligation, of course.

☐ Also send information on Sparling's line of Water Control Equipment.

Name _____

Company _____

Address _____

City _____ Zone ____ State _____

Send to: Dept. JA-2,
SPARLING METER COMPANY
225 N. Temple City Blvd.
El Monte, Calif.

TIGHT IN THE JOINTS?
YOU BET!

Getting tight joints is simple—USE STEEL PIPE. Steel water pipe *assures* bottle-tight joints for the life of the line. When you have steel-tight joints, there is no need to over-design water pipe lines to take care of costly needless water waste. Tight joints cost less to begin with—and actually save you money through the years. It's another good reason why you're so smart to specify steel pipe.

"WHEREVER WATER FLOWS—STEEL PIPES IT BEST"

STEEL PLATE FABRICATORS ASSOCIATION

79 W. MONROE ST., CHICAGO 3, ILL.

Select the BEST for SURE Operation and LOWER Maintenance Cost

ROCKWELL — "AWWA"

Butterfly Valves

- Drop-Tight Shutoff
- Minimum Restriction to Flow
- Minimum Pressure Drop
- Non-Clogging
- Better Control—Manual or Automatic
- Compact—Lighter Weight
- Reduced Installation Space
- Lowest Installed Cost
- Less Maintenance

Write for Bulletin 574

Made in all standard sizes of cast iron, cast steel, stainless steel, bronze and other alloys; natural or synthetic rubber seat with clamping segments, or spool type rubber liner extending over flange faces. Compact, rugged construction. Manual or automatic control for convenient, easy operation.

ROCKWELL "D-D"

Diaphragm Valves

For controlled throttling or cut-off on corrosive or abrasive fluids, or water. Disc for positive closure, diaphragm for positive sealing. No packing gland to leak. Oversized ports; high capacity; low pressure drop. Lower handwheel torque and fewer turns to close. Lower thrust for automatic operation. Body of any metal or with any lining; disc and diaphragm of rubber or plastic. Sizes—½" to 6". Write for Bulletin.

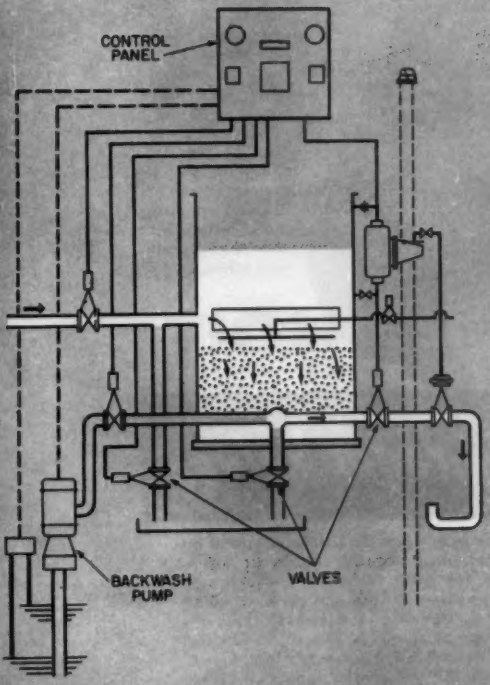
W. S. ROCKWELL COMPANY

2608 ELIOT STREET • FAIRFIELD, CONN.

New Valveless Water Filter

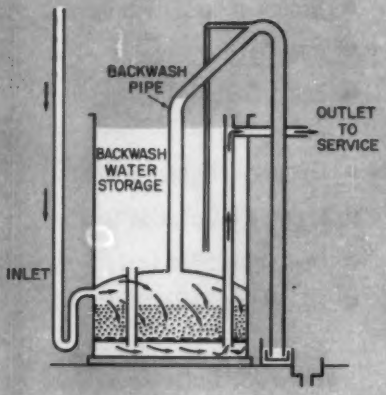
CONVENTIONAL gravity filter and hydraulic controls for automatic operation. Approximate installed cost of a unit that will filter 500,000 gallons per day:

$21,700.00



NEW PERMUTIT VALVELESS Gravity Filter. Completely automatic operation. Approximate installed cost of a 500,000 gpd unit:

$11,900.00



HOW IT WORKS: As dirt collects on the sand, increased head pressure slowly raises the water level in the large backwash pipe. See diagram. When it spills into the downward section of the pipe, it starts a siphon (backwash) flow that draws water down from the Storage area and up through the sand, "floating" dirt off to waste. When backwash water is gone, air enters the small tube and stops the siphon. Flow through the sand reverses and the first filtered water (rinse water) goes to Storage area until full. All flow then runs to Service.

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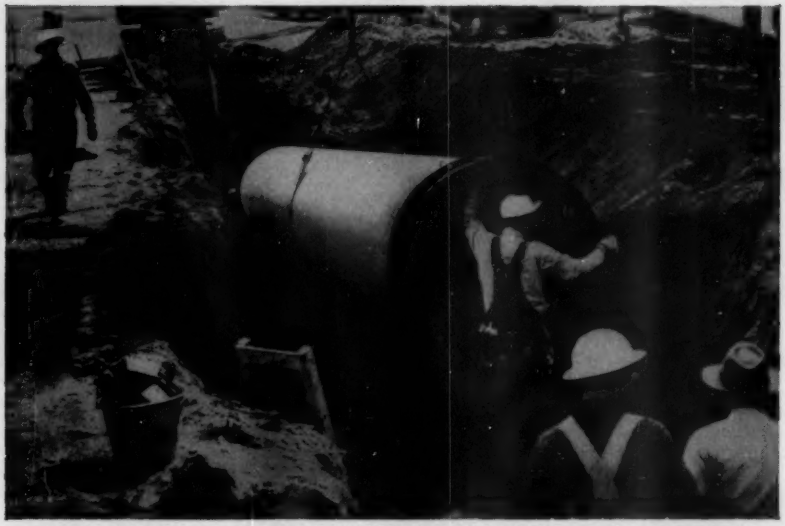

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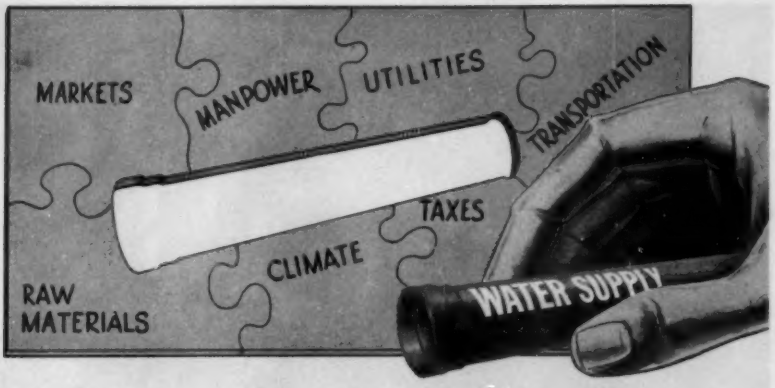

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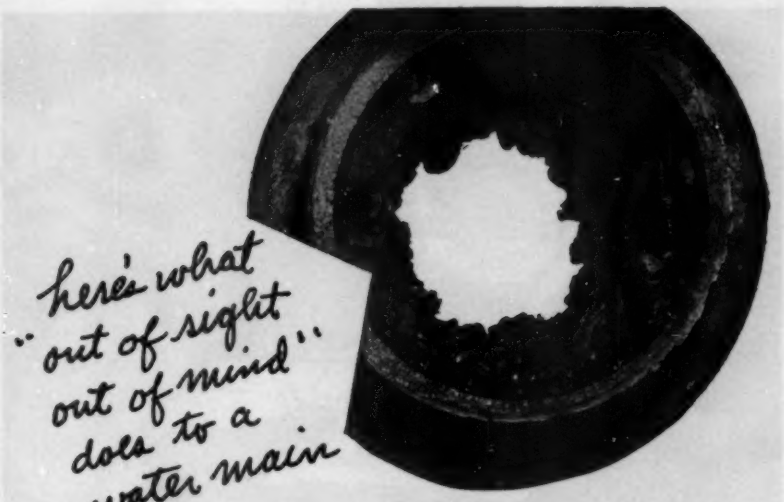

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Principles of Personnel Management for Water Utilities

Robert S. Millar

A paper presented on Sep. 25, 1957, at the Rocky Mountain Section Meeting, Santa Fe, N.M., by Robert S. Millar, Secy.-Mgr., Board of Water Comrs., Denver, Colo.

THE water utility field can do much to improve its status as an important segment of communities of all sizes. Regardless of ownership of the utility, public or private, it can give the community the improved service rendered by qualified personnel proud of the jobs they are doing. When this is accomplished, water utilities will have little trouble recruiting good people for jobs which will challenge their abilities.

The unbelievable advances in the sciences which have produced fission and fusion of the atom and made possible destruction of this world itself; advances in the field of medicine which have increased the life span of man and promise to increase it even further; advances in industrial techniques from development of standards of measurement, interchangeability of parts, and mass production to electronic computers, which some believe have brains; advances in psychology to the point where many now believe that the long-held secrets of the human mind and its functioning are being understood and even measured; all these advances have one common denominator, one thing in common, man. And man is the subject matter of, the material of, personnel. This field of activity, study, and research called personnel is, in the final analysis, man, just people, many different kinds of people. The personalities, character, education, and experience of people produce any advancement that is made.

Selection, training, and utilization of manpower are the fields in which continuing improvement must be made if the goal of truly efficient public service is to be attained. There have been many improvements in pumping methods over the past few decades. Many can remember the use of direct-acting, reciprocating steam engine pumping

units and hand fired boilers. Recording devices now tell exactly what is going on miles away, and radio dispatching clears up trouble spots in the distribution system before they develop enough for customers to recognize changes in service conditions. Office procedure has changed also. But how much change has taken place in the personnel procedure at your plants?

Many so-called new methods or procedures are old models in new clothes. And there are many simple, old-fashioned, fundamental truths that should never be forgotten or overlooked in an effort to "get up-to-date." Personnel policy affects human lives: the hopes and aspirations; the fears, worries, and doubts; the achievements, accomplishments, and failures—in short, the happiness and joys or the depression and sorrows of people. A good personnel program is one which improves the morale of loyal employees.

The basic principles of good personnel administration will vary in degree with the size of plant, complexity of operations, and other factors, but the principles are fundamental and apply to all. The purpose of this article is to discuss some of these principles of good personnel management.

Indoctrination and Selection

All personnel policies and practices should be reduced to writing and made available to all employees. A well prepared employee's manual is the best way to tell employees about their jobs, what is required of them, the benefits they enjoy, and something about the company for which they work. The manual should answer the many questions an employee might want to have answered about his job. It should not be just a book of rules, it should help morale by proving that the company or department is interested in employee happiness and well being. But the best written manual is no substitute for the personal interest of the employer.

Selection of employees should be based on a thorough investigation of the applicant's aptitude, training, experience, and character. The material from which utilities build an efficient staff is, of course, the new employee. The cost of selection and training of poor or even mediocre employees on a "we will try him out" approach is so high, no one can afford to continue on that basis. Time and money invested in establishing and using a good selection procedure will pay dividends in increased efficiency and reduced turnover. The average form sent to previous employers is too frequently not a very reliable source of honest information. Contacting the previous employer or employers personally, rather than by telephone, produces much better results than merely sending a form. A good, honest, and complete evaluation of the aptitude, training, experience, and character of an applicant is no substitute for personal experience with an individual. A period of probationary employment affords the opportunity of measuring the accuracy of estimates made. If the temporary employee, or probationer, proves to be inadequate for the job, or any other job which is open, the situation should be recognized and faced realistically, both for the good of the employer and the prospective employee. The investigation period should always include the period of probationary employment. Every employee should be told his duties, responsibilities and areas of authority, the conditions of em-

ployment, rate of pay, working hours, and benefits. This he should learn in detail at the time of employment.

Records and Evaluation

Adequate records, including the application blank, the employee's history record, status changes, and personnel evaluation should be maintained for each employee. If he leaves for any reason, an exit interview should be made a part of the record. It is very easy to develop an extensive records program that serves little useful purpose. This does not mean that records are not important, but simply that they must contain useful information. The records each water utility keeps should be designed to record the information that will be useful to the management of that plant. The information on any particular form should depend upon the use to which the form will be put. The series of steps that could be used in developing a form could be:

1. A detailed description of the objectives the form is expected to achieve is drawn up.
2. The basic form can be developed from the description and checked against it.
3. Before the form is finally printed, it should be used in mimeograph form long enough to assess its value.
4. Where the value of the form has been proved, it can then be printed in quantity.
5. A periodic review of all forms should be made to determine their efficiency. And if they can be improved, there should be no hesitation in making changes to eliminate unneeded data, to add valuable information, or to eliminate the form completely.

Every plant should set up standards of worker performance based on a job evaluation. This evaluation should be discussed thoroughly with the worker at the time of employment. The standards thus developed may be used as a basis of merit evaluation. An analysis and evaluation of every job in the plant is the basis of any sound personnel program. Job evaluation is an effective method of determining the duties and responsibilities of each job. A thorough evaluation should establish and maintain a logical relationship among the duties and responsibilities of positions, the qualifications to fill them, and the salaries paid. The job evaluation may be performed by the individual utility's own administrators, or an outside expert may be used. Job evaluation is not a job for an amateur; it should be undertaken only by a person who has had experience in this field. If an outside firm makes the evaluation, a staff member of the utility should work as closely as possible with the consultant to insure an intelligent administration of the plan once it is established. In order to maintain the usefulness of a job evaluation plan, it must be reviewed constantly to see that it is up-to-date and that all employees are properly classified.

Employee Grievances

Employees should be encouraged to discuss their grievances and complaints with their supervisor. An open-door policy on the part of the plant manager or the personnel office is a poor and ineffective substitute for a well planned grievance procedure. A city or a company has some personality, but it takes the live relationship of honest interest expressed by an indi-

vidual to reach people most effectively. It may be the foreman, superintendent, manager, elected official, or corporate officer who can best reach the new employee. Someone must take time to talk things over, explain details, and answer questions. Management that waits for employees to come to them with complaints and grievances will often find a union, a lawyer, or some other group representative bringing in these complaints rather than the employee himself.

The first link in the grievance procedure is the front-line supervisor. He must be prepared to listen to and answer employee questions, or any effort to develop a grievance procedure will fail. The manager must make it clear to all foremen that grievances are to be treated seriously and that the employee is entitled to prompt and accurate counseling. Everyone must learn that grievances are not necessarily a reflection upon the foreman's ability. Good foremen should be able to settle most of the grievances that arise in their departments. It is important that an accurate record of all grievances and their disposition be kept. This record will serve as a guide to modify conditions and personnel regulations and as the basis upon which appeals for the judgment of the foreman can be decided.

The next step in the grievance procedure will vary with individual water plants. Some may use a joint committee plan or a union committee; others may use the personnel officer or the plant manager to receive appeals from the foreman's judgment. Whoever receives these appeals must remember that every employee is entitled to courteous consideration and an impartial decision. Some water plants are located in cities which provide a special board of review, such as a civil service or career service board. If the water plant management has made an honest effort to solve grievances, there will be few grievances carried to this sort of a board.

Promotion and Supervision

Employees should be informed of the organizational structures so that they may prepare themselves for promotion if they should so desire. Promotion should be made from within the organization wherever possible. A definite promotion policy has several advantages: it increases efficiency, improves worker morale, and tends to attract better qualified applicants in the lower jobs because of the opportunities afforded to make the water industry a career. A promotional plan must be tied in with the overall personnel program. If adequate employees are not hired and proper training not provided, any program of promotion from within will fail. If the overall personnel program is operating properly, there should be no problem in selecting promotable employees from the ranks. The procedures used for selecting the best possible candidate for promotion will vary from plant to plant and job to job. A combination of four factors can be used to make this selection. These factors are: seniority, competitive examinations, the judgment of the supervisor, and merit ratings. The exact method to be used must be worked out by the responsible member of management. Transfers from job to job can be of value to both the plant and the employee. They can be used as a training device or to help employees to find jobs better suited to their abilities. Uncontrolled transfer-

ring, however, can cause serious damage to efficiency and morale. It should be the duty of the responsible administrative officer to develop controls that will insure the value of any transfer.

Every employee should receive adequate supervision. This means, among other things, fair personal treatment, constructive criticism, praise for work well done, a willing ear to hear him, freedom from prejudice and favoritism, technical instruction, and delegation of responsibility. It is well to remember that once the basic outlines of an efficient personnel program have been developed by top management, its success depends almost entirely on the way it is administered by the lower levels of management. The best plan of selection of new employees will be wasted if they are not properly trained; the best grievance procedures will fail if the foremen do not use them; and the most accurate job classification plan will be useless if it is not followed. Top management and the personnel office should do everything in their power to see that all foremen and supervisors are informed and trained to administer the personnel program. Civil service, employees' committees, attorneys representing employees, and unions are systems, devices, personalities, and methods used to carry the employee's story to management, and they are believed to be the protection employees have against unfair or unscrupulous practices of management. When any of these employee methods, or any of those of management, contribute toward weakening an organization, corrective measures should be taken to assure improvement in the operations and services of the water utility.

Pay Plan

A pay plan in which positions are evaluated by classes in relation to one another and a pay scale developed for each class of position is the first step in assuring fair employee compensations. Such a plan must be governed by a set of rules authorizing and controlling the pay, and changes in pay, of each class of position and those who are so classified.

Several steps are necessary in developing the pay plan:

1. Each position should be analyzed and, on the basis of the complexity and responsibility of the position, ranked with others in order of importance.

2. Salary data collected by a survey of comparable community or area wage rates can then be compared with each classification, or as many as possible. When comparable jobs do not exist, the wage rate should be determined from other job rates paid for similarly rated classifications.

3. Semiannual or annual reviews of the relationship between the wage schedule and community wage rates should be completed to assure a fair wage and salary level.

4. Reviews should include studies of fringe benefits allowed in the community or area.

Conclusions

In the final analysis, the community deserves the best service possible for the money paid. The qualitative and quantitative measurements of the service rendered are determined by the measure of those people who are the personnel of the department or company. The character, personality, and experience of those responsible for water service to a community should

be among the highest to assure continuous, healthful, and safe service. The personnel requirements and rewards should be such as to attract to these important jobs of public service people who will conscientiously measure up to high standards.

These principles should be treated as guideposts only. They point the way. They do not locate the stumbling blocks or other hazards of the path to be followed. Here again, it is man who must guide every step. No fixed rules should hinder the minds of those who guide the way to better results. Communities deserve the best possible results which can be obtained. It is for men in the water utilities to prove themselves equal to the task, equipped to meet the challenge and capable of producing their service in the most efficient manner possible.

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Attracting and Retaining Competent Personnel for Water Utilities

William T. Ingram

A paper presented on Apr. 11, 1957, at the New York Section Meeting, Elmira, N.Y., by William T. Ingram, Cons. Engr., New York, N.Y.

THE question of whether the water utilities industry is able to attract and retain competent personnel is rhetorical. The industry must do this. The job of supplying water is in the hands of personnel who have a responsibility to operate the entire system so that safe, potable water is available for use 24 hr every day. These persons must be competent.

The purpose of this discussion, then, is to suggest some of the more important reasons personnel are not easy to retain in water utilities and to offer some ideas about what may be done to counteract these adverse influences.

New York State has over 900 water systems varying in size of staff from the part-time executive and a maintenance crew of one or two to the giant of the industry, New York City.

Small Systems

The median population served by systems in the state is about 1,000 persons. At least one-half of the problems of staffing are those of the small systems of villages and towns. Some of the problems faced by water utility personnel operating these systems are:

1. Governing boards are preoccupied with many matters other than water utilities. It is only when complaints filter back to board members that there is real concern and then only for the cause of complaint and its correction.

2. Utility crews may have more than one duty. Such crews may have multiple assignments, including sewer maintenance, highway maintenance, refuse disposal, and others in the area of public works functions.

3. Time available for constructive planning to reduce maintenance and to improve the system is very little. If taking such time means an increase of staff, the activity is looked on with question and even disfavor.

4. Even planning and carrying out necessary procurement of equipment for day-to-day replacement, repair, and improvement brings frustration. The discussions on purchase at times reach no decision, and ultimate authorizations of work are slow.

5. Resource to a village or town engineer or a consultant on water supply is too often limited. There may be no full-time engineer, and there may be strong board resistance to the use of a consultant.

6. The opportunity to discuss water problems and to learn more about the job of obtaining and suplying water may be curtailed. Time spent in visiting the neighboring town, or attending a water supply meeting is too often impugned as a junket by those who fail to realize the real needs for

exchange of knowledge and experience in the field.

7. Salaries are more often low than high. To the uninformed, almost anyone could operate a water system. The dollar advantage of skilled and well planned operation over the "try it and see if it works" tactics employed on more than one occasion are not easily computed if there has been no experience with the former method. Thus, the well trained water utility specialist has some difficulty in commanding a salary corresponding to his responsibility and ability.

8. There is usually no future advancement within the utility staff. The opportunities for a competent executive comprise similar appointment at another location or advancement into one of the higher salaried public works positions at home or elsewhere.

Intermediate Systems

The group of water systems of larger size, but still relatively small in numbers of personnel, has perhaps more pressing problems. Systems that have been serving areas with 10,000–20,000 persons are suddenly being placed in the position of providing service for doubled or greater populations, expanding distribution lines, reinforcing networks, procuring additional sources, increasing treatment facilities, and coping with the operating problems of inadequate facilities brought on by increased demands. The situation can become untenable for water utility management and staff before the governing board is willing to recognize the existence of the problems.

The system is no longer a small staff operation. Yet the town or village board is slow—too slow—in recognizing the changes taking place and overcautious in conceding that responsibility for planning, construction, operation, and maintenance is important enough to require first attention. Utility staffs of competence are now mandatory. It becomes critical under these circumstances to have well qualified supervisory personnel who are provided with reasonably solid authority to plan and carry out the work of the water system.

Personnel problems arising among this group, between small and large, are representative of those found in hundreds of systems over the country. It is, therefore, worth while to examine some of the criteria that influence an individual to take or remain on a job in a system of that intermediate group.

1. The job must be interesting and satisfy the individual's need to use his background of training and experience.

2. The salary must be enough to live on and meet family, community, and personal security requirements.

3. Job security must be acceptable. The attractiveness of pension plans, health benefits, medical care plans, and similar substitutes for higher salary is a factor. Perhaps of greater importance is the longevity and tenure of appointment. Few men are attracted to a position from which a political change may oust them.

4. Future prospects of working with the utility should be favorable. Pleasant working conditions with compatible people and a reasonable anticipation of advancement are part of that future.

5. Administrative supervision should be such that frustration does not become a prime focus of attention and a principal point of discussion.

6. Prestige, both personal and community, should be above question. The family reaction to the work has to be

favorable, and the acceptance of position as desirable by the community is essential.

7. Reasonable resources and opportunities for introducing ideas to the work should be available. Access to training courses and conferences and recognition of good work on the job are most desirable.

8. There should be adequate staffing so that individuals do not have to contend with either excessive physical or mental effort in an attempt to meet responsibilities that are taken seriously. A man's desire to work long hours over extended periods of time is soon ended if it becomes clear that advantage is taken of that desire.

Management Responsibility

It is recognized by those of the water utility field that operation of a water system is no sinecure. There is, however, a management responsibility to review its own position. Management extending to the core of governing boards could profitably examine again and again the above precepts to evaluate specifically how it rates. Although there is no one best answer determining actions that may be taken by management, these general points are suggested as a guide for determining what may be done to attract and hold competent personnel.

1. Avoid delay in readjustment of salary levels commensurate with responsibility.

2. Recognize position and administrative lines with appropriate titles.

3. Exercise every effort to establish prestige for the water utility man.

4. Be certain that training, experience, and abilities of key personnel are equal to the job responsibility. Unable persons holding responsible positions can deliver only irresponsible performance.

5. Encourage on-the-job training.

6. Determine and provide work incentives.

7. Be sure that talents of the staff are not being misused.

8. Encourage and promote a spirit, teamwork, and unity that will build a department into an organization engaging in routine or emergency action with effectiveness.

9. Simplify and eliminate, if possible, red-tape practices required to authorize work or purchases. Recommendations of department heads should be acceptable, since the head is (or should be) the person who knows what is needed.

In the last analysis, there are few men in the water utility field who are unhappy and discontented with their position, if the water is the best quality obtainable, is adequate for every need, and runs in a system that is in good repair. When executives have the authority, financing, and staff to keep the system up to provide water of excellent quality, they usually have an understanding and progressive top-management board or council that has taken most of the suggested actions. Such system organizations have a good reputation, attract personnel, and hold them.

Artificial Recharging of Water-Bearing Formations

O. J. Muegge

A paper presented on Sep. 4, 1957, at the Wisconsin Section Meeting, Milwaukee, Wis., by O. J. Muegge, State San. Engr., State Board of Health, Madison, Wis.

THE subject of recharging water-bearing formations is broad in scope. This article, however, will be limited to a discussion of artificial or induced recharge and essentially will omit reference to natural recharge.

The artificial recharge of ground water is practiced in many areas of the country. In many instances the recharge of ground water formations is incidental, the motivation for such recharge being: [1] the disposal of waste waters, [2] the drainage of low areas and storm water, [3] the impoundment of surface waters and control of flood waters, or [4] the irrigation of agricultural lands. Although recharge of aquifers from such operations is incidental, such recharge in the aggregate would be a substantial percentage of the volume of ground water used.

Incidental Recharge

Incidental recharge of aquifers resulting from the disposal of liquid wastes primarily affects the unconsolidated formations and is most effective in the areas in which the soil is pervious. In some areas, however, consolidated formations, particularly limestone, are also utilized for waste disposal. Wastes disposed of by means of soil absorption include sewage, principally from residential and commercial units in areas not served by public sewers, and wastes from various types of industry. Disposal by seepage pits, tile fields, lagoons or stabilization ponds, and spray irrigation all function in recharging unconsolidated formations. Data are not available to permit a reliable estimate of the total volume of water recharged through use of soil disposal systems in Wisconsin.

Recharge resulting from the drainage of low lands or disposal of storm water is accomplished by the installation of wells. At one time many so-called drainage wells were installed in Wisconsin and other states. Experience soon indicated, however, that where the wells extended into unconsolidated formations or sandstones they were short-lived, because of the plugging of the formation by the silt and suspended matter entrained in the surface or drainage waters. Where such drainage wells penetrate limestone formations of a creviced nature, such as exist in many sections of the country, they do function for a considerable period of time. Many of these wells have a high disposal rate, one in Florida (*1*) operating at 9,500 gpm. Use of wells for disposal of drainage and waste waters is not allowed in Wisconsin.

The impounding of water, such as is practiced in the reservoir systems in

the northern part of Wisconsin, results in the recharge of adjoining lands as the reservoirs are being filled following high periods of runoff. In Wisconsin such recharge is mainly beneficial to the operators of the water storage system, since the ground water so recharged amplifies the quantity stored and the amount of water that can be withdrawn from the reservoir system for subsequent use. In other areas, where water use has caused ground water levels to drop, such impoundments could add materially to the available supply. Soil erosion control and the spreading of flood waters over sandy valley areas, as practiced in some western states, will also replenish ground water resources (*2, 3*).

Irrigation of lands by water withdrawn from surface sources will tend to augment ground water supplies, provided the formation is not already at saturation level. The benefit derived from irrigation practices would vary with the type of soil, method of application of the water to the land, and the degree of control exercised by the irrigator. As much as one-third of the applied water could enter the ground water supply.

None of the incidental recharge practices mentioned are of great importance in increasing the availability of ground water for municipal and utility use in Wisconsin, since such recharge operations are largely limited to areas where no shortage of water has been demonstrated. In fact, in some areas the disposal of wastes by soil absorption into both unconsolidated and consolidated formations has resulted in a deterioration of ground water quality, bacteriologically and chemically. Although most of the wells adversely affected by such disposal are privately owned, some public utility wells have been involved. For instance, in Wisconsin the use of an old well for drainage of waters from a display fountain caused contamination of the utility well located 2,400 ft away. In Green Bay, an abandoned well used to dispose of laundry waste and basement drainage contaminated a city well, 600 ft distant. In 1929, the relocation of a river channel in Fond du Lac over an unsealed well resulted in the accidental recharge with contaminated water of a limestone formation supplying nearby utility wells, causing a serious outbreak of waterborne disease. Similar experiences can undoubtedly be cited in other states where creviced limestone formations exist.

Induced Recharge

Induced recharge of water-bearing formations results when the withdrawal works are so located and designed as to increase the natural rate of recharge from the surface. Water systems bringing about induced recharge can perhaps be found in most of the states of the union. In Wisconsin, only a limited number of installations exist at this time in which induced recharge is a substantial factor in ground water productivity. The best current installations utilizing induced recharge are the two radial collector wells at Manitowoc, both of which are located and constructed so as to induce recharge from Lake Michigan. An older water supply installation at Superior is of a similar type, being an installation of well screens horizontally below the bottom of Lake Superior. In former years infiltration galleries lying parallel to streams or lakes at Hayward and South Milwaukee were used as sources of supply, and a similar installation is still in use at Nekoosa. A completely artificial infiltration system also formerly served as the source of supply

for Black River Falls. In other communities, wells have been located near lakes and streams with the thought that the yield of ground water could be improved. Unless the stream or lake bottom is sandy, however, and is renewed through scouring, redeposition, and wave action, no material benefit results from such locations.

Applied Recharge

Applied artificial recharge of aquifers is being practiced or studied in a number of states for the purpose of developing barriers against salt-water encroachment and to augment natural and induced recharge. Salt-water encroachment results from the overpumping of aquifers along the sea coasts, from heavy pumping of aquifers underlain with saline water, from improper construction of wells extending into saline waters, or from improper disposal of oil field brines.

Various methods of combating sea water encroachment have been used in California. At Camp Cooke, near the mouth of Santa Ynez River, a weir was constructed across the valley with its crest 5 ft higher than high-tide level so as to increase recharge to the well area and to prevent tidewater intrusion (*4*). Recharge basins at El Segundo and Redondo Beach were tried for the purpose of building up a fresh-water ridge (*5*). Because of adverse conditions encountered in the basin area, the use of diffusion wells was next studied and found satisfactory for production of a water barrier if the wells were spaced at 500-ft intervals and the water was properly conditioned (*6*). To be effective, the mound of water created must obviously be sufficiently high to counteract the greater specific gravity of the sea water and to provide for a movement of fresh water inland.

On Long Island, sea water encroachment was reduced by a requirement that owners of all wells pumped at a rate of 69.4 gpm or more return the used water in an uncontaminated condition into the ground (*7*). Some of the diffusion wells have been in service for over 20 years, maintenance work, however, being required during the period. All of the diffusion wells return water to the unconsolidated formations and usually to the strata from which the water was withdrawn. A rise in the temperature of the ground water, as much as 20° in some wells, has been observed (*8*).

In Brooklyn, N.Y. (*9*), El Paso, Tex. (*10*) and Camp Peary, Va. (*11*), brackish water wells have been recharged with fresh water. Studies of these operations indicate that there is but little intermixture of the fresh and saline waters and that nearly all of the recharge water can be subsequently withdrawn. These studies further show that aquifers containing highly mineralized water can be made useful for storage of fresh waters.

In various inland areas of the country, aquifers with highly mineralized water underlie waters that are only moderately mineralized. In some cases the highly mineralized aquifers are under artesian head greater than that in the upper formations. Therefore, when a well taps both upper and lower formations, contamination of the upper formation results, particularly when pumping of adjoining wells lowers the water level. Such detrimental effects can best be avoided by constructing wells so that they tap only one of the formations and by using watertight construction, relatively permanent in nature, through the upper formation when the lower horizon is to be utilized. In such event, the mineralized

water should be returned to the aquifer from which it is withdrawn to prevent contamination of upper formations or surface waters. Return of saline waters in such manner is practiced in inland oil fields for a two-fold purpose, namely to dispose of the brines and to maintain pressures in the oil-producing formations so that more oil can be recovered than would otherwise be possible (*12, 13*).

Artificial recharge is being used in many places throughout the country for the purpose of augmenting natural or induced recharge. Such recharge is attained by various methods of spreading surface water and through use of diffusion or recharge wells. Methods of spreading water for recharge include the flooding of areas by means of ditches, the use of ridge and furrow systems, the use of basins, and the use of impoundments with or without controlled stream flow. Recharge operations by spreading are most extensive in the western states, and particularly in California (*14*). Proposals have been made to use reclaimed sewage for recharge (*15*). Spreading by basins or impoundments has also been used extensively in New Jersey, including, among others, the installations at Perth Amboy, Princeton, Lake Mohawk, and East Orange (*16*).

In the central states the better known recharge operations are those at Des Moines, Iowa, and Peoria, Ill. Recharge operations at Des Moines were started before 1922, and they comprise the use of basins to augment induced recharge to a collecting gallery parallel to Raccoon River (*17*). As in some of the western installations, the bottoms of the basins are deeply scarified as needed to maintain adequate recharge.

The recharge project at Peoria (*18*), also by means of prepared basins, was undertaken in 1951 for the purpose of studying recharge by spreading and to amplify the well supplies for the water utilities and industries. The recharge operations are carried on only during the cooler period of the year when the river water temperature does not exceed 60°F. It was established that the cool recharge water remained on top of the existing ground water. A number of other hydrological, bacterial, chemical, and physical determinations have been made, but it was found difficult to explain all of the variation in recharge rates on the basis of recharge heads or water quality. Studies are being continued by the Illinois State Water Survey.

Recharge of aquifers by means of wells is not in widespread use. The longest experience in this regard is that of Long Island, previously reviewed. Mention has also been made of drainage wells, the California diffusion wells, and those of El Paso and Camp Peary. Well recharge with city water was tried at North Hollywood with poor results (*19*), the formation becoming sealed after a few days of use because of movement of fines in the formation or turbidity in the water.

Another well-known recharge operation is that conducted in the Louisville, Ky., area (*20*) where an aquifer was being pumped by industries seeking cool water at a rate 20 mgd in excess of natural recharge. In 1944, steps were taken simultaneously to stop pumping during cool weather, to recharge the aquifer with purified city water during such a period, and to conserve water by economizing on use. In this situation, the production wells are used for recharge. The results have been satisfactory and have enabled the large industrial users to pump cool water during the summer months when city water is

too warm. Storing of cool water during the off-peak season in the underground formations for use during peak periods has also been found successful at Binghamton, N.Y. (*9*) and Lubbock, Tex. (*21*). The question arises whether or not similar recharge operations may not be practical and desirable in Wisconsin.

In Canton, Ohio (*22*), collector-type wells tap both of two water-bearing gravel layers that are separated by a clay strata. The top layer is readily recharged locally but is thin and can be exhausted rapidly by heavy pumping. Therefore, water is allowed to pass from the upper to the lower gravel layer freely, total recharge being about 8 mgd through the three collector units. One collector is equipped with pumps delivering 10 mgd from the lower aquifer.

A somewhat similar recharge process is going on steadily in the Milwaukee area. Many wells here are open to both the Niagara dolomite, above an impervious shale layer, and the lower sandstone formations. Because of the approximately 70-ft difference in water levels in the upper and lower aquifers, drainage from the Niagara dolomite to the lower lying St. Peter and Upper Cambrian formations takes place, except when such wells are being pumped. In 1950, Foley, Walton, and Drescher (*23*) estimated that 5.5–6.0 mgd was being recharged into the lower formations. Similar conditions could exist in some other areas in the state where the lower formations are overpumped.

Wisconsin Conditions

It is to be noted that except for drainage wells, oil wells, and the Milwaukee area wells, the recharge projects discussed herein deal with water problems of sand and gravel formations. No serious problems as yet exist in Wisconsin involving unconsolidated aquifers, other than those in the central section where permeable formations are lacking, are limited in area, or cannot be readily recharged from the ground surface. With few exceptions, it would appear that recharge by spreading could not be applied practically at this time to solve the requirements of agriculture, industry or utilities of Wisconsin. Recharge by diffusion wells, however, might prove to be of value, particularly in connection with depleted consolidated formations or highly mineralized water producing aquifers. The element of cost of producing or providing water suitable for diffusion through wells, the need for cool or other special quality water, and the probable saving in cost of operating wells would need to be favorable to make well recharge a practical project.

Proposed Research

Discussions have been held over a period of years in Wisconsin on the desirability of studying the problems and effectiveness of recharge of the sandstone formations in the Milwaukee area. Some preliminary work was done 8–10 years ago to determine the feasibility of recharging these aquifers with Milwaukee city water during the winter off-peak season. These investigations made in cooperation with the city water department disclosed that the lake and sandstone waters did not react detrimentally. The research project was not carried further at that time because of financial and personnel limitations, early budget requests for funds not being approved. In 1955, however, the legislature did appropriate funds for employment of an engineer for the study who, because of re-

cruitment problems, did not become available until July 1956.

Since his employment, library research has been conducted to review problems related to ground water recharge, and conferences were undertaken to arrange for use of wells for recharge, for use of Milwaukee city water as the recharge source, and for financial support by industry. Some roadblocks were encountered, which necessitated detours, but much progress has been made. Prospects are favorable that a study of recharge, using one of the wells of the city of Wauwatosa, will be carried on shortly.

The purpose of this study will be to determine: [1] the rate of discharge in terms of well production per foot of head; [2] the head recovery; [3] the recharged water recovery; [4] the method by which diffusion can best be determined; [5] the effect of injection water temperature; [6] the cost of recharge; and [7] the physical factors effecting recharge operations. This study will be conducted in full cooperation with the staff of the Federal-State Cooperative Water Level Study Project.

In addition to the recharge study, it is planned to keep in close touch with studies of natural recharge in the Green Bay area, where a purified supply from Lake Michigan has recently been introduced to replace the overpumped artesian well water supply. Artificial recharge studies in this community are also contemplated as a future project.

In addition to the above projects, it is proposed that a study will be made of water use and reuse by industry and of the problems that may be associated with the conservation of cooling waters by recharge to the ground water. In this connection, certain provisions of a bill which had been proposed by the Wisconsin Natural Resources Committee, dealing with recharge, read:

> The board may require that all or part of the water withdrawn from such wells, or an equivalent volume of water from some other source, be returned to the aquifer or aquifers from which such well or wells withdraw water or to an alternate aquifer, with such purification, treatment, and safeguards as are essential to prevent such recharge from deleteriously affecting the quality of underground waters.

It is believed that such a provision, or one similar thereto, should be a part of any comprehensive water supply law.

Conclusion

In conclusion, it may be pointed out that other communities and other states have ground water problems that are as troublesome and in most cases more acute than those of Wisconsin. Research leading to solutions of the problems of overdraft will be in keeping with progress and studies in this field throughout the country. Water utility personnel should participate to the fullest extent possible in arriving at practical solutions of water use and of conservation of this most essential resource.

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Geophysical Techniques in the Location of New Sources of Water

Robert F. Spanski

A paper presented on Sep. 4, 1957, at the Wisconsin Section Meeting, Milwaukee, Wis., by Robert F. Spanski, Cons. Geologist, Springfield, Ill.

GEOPHYSICAL methods of exploration consist of the measurement and identification of changes in certain physical properties of the earth at or near its surface. At present, the geophysical methods most applicable to ground water prospecting are the seismic and electrical-resistivity methods. They involve, respectively, the measurement of the ratio of propagation of elastic waves in the earth and changes in the electrical resistance of the earth. These two methods were primarily developed for use in prospecting for petroleum and minerals. Adaptations and modifications of these basic methods, however, are extensively used for engineering purposes and ground water prospecting.

These techniques are especially well suited for reconnaissance exploration of large areas. When used in conjunction with geologic data, the methods, under favorable conditions, give results which indicate the character and sequence of strata and the depth to bedrock. But it must be emphasized that the full value of these methods are obtained in their correlation with, and as supplements to, other data.

Seismic Method

The seismic method utilizes the elastic properties of the materials of the earth. The velocity of propagation of a wave or impulse in an elastic body is a function of the modulus of elasticity, the density of the material, and a combination of other factors. Therefore, there exist great differences in wave velocities in solid rock and loose sedimentary material.

The energy from the explosion of a small charge of dynamite in a shallow borehole is propagated through the earth materials by elastic waves advancing outward from the point of explosion. These elastic waves consist of longitudinal or compression waves, transverse or shear waves, and various types of surface waves. Some of these waves are reflected at the interface between beds, and some are refracted in passage through the media. It is these refracted waves which are used in near-surface seismic work. In passing across a boundary of different materials the elastic wave is refracted toward the plane of the boundary when going from a low-velocity layer to a high-velocity layer. The refraction is toward the perpendicular to the boundary when it passes from one layer to another with a lower velocity of wave propagation.

The equipment and instrumentation required for refraction work is compact and portable. Basically, the equipment consists of:

1. An amplifier case, with twelve amplifiers, balance potentiometers, and test circuitry
2. A recording oscillograph, with twelve galvanometers and a synchronous motor on the paper drive
3. A power supply
4. A blaster to fire the shot and provide accurate timing between the interval of detonation and the starting of the paper drive motor in the recording oscillograph
5. Seismometers and cables.

In field operation the seismometers are usually laid out with the shot point at one end of the spread. The length of the spread, or the distance from the shot point to the farthest seismometer, is three to twelve times the desired depth of penetration. At the shot point an auger hole is dug and charged with from one to three sticks of dynamite. The shot is fired, and the time interval between detonation and arrival of the first impulse at each seismometer is transmitted to the amplifiers and recorded by the oscillograph on a moving strip of photographic paper, on which are marked time intervals of 0.01 sec.

Only the arrival times of the initial impulse at each seismometer are used. By plotting the travel times of the impulse as a function of the distances between seismometers and shot point, drawing straight lines through the plotted points, establishing intersections of these lines, and then computing velocities in the zones thus established, depths to bedrock or any velocity discontinuity can be determined.

It should be emphasized that the seismic method can be used only when the velocity of the wave becomes greater in each succeedingly deeper layer. If a low-velocity layer occurs beneath a high-velocity layer, neither its presence nor thickness can be determined by this method. Complications are also encountered when the wave velocity in a thick overburden increases only gradually with depth. The path of the first impulse and the travel time diagram will then be curved, and it may be difficult to determine the actual wave velocities and thickness of the nonuniform stratum. With inclined layers only an average depth is determined from the travel time diagram, and it becomes necessary to reverse the position of the shot point in the spread in order to determine the actual depth and amount of dip along the shot line.

Electrical-Resistivity Method

Of the electrical methods applicable to ground water investigations, the electrical-resistivity method, based on differences in resistivity of the subsurface materials, is the most widely used. An electrical field is produced in the ground from two field electrodes, and the drop in potential is measured between two potential electrodes.

The conductivity of earth materials varies from 100 billion ohms per centimeter when dry to several hundred or several million ohms per centimeter when saturated. It is apparent, therefore, that the electric current flows by ionic conductance. The fact that the electrical resistivity of soils and nonmineralized rocks is dependent on the electrolyte content of the fluid and the porosity of the soil is of particular importance in work in alluvial sediments. Near the surface the pore filling is mostly air, the resistivity will be very high, and the passage of the current will almost entirely depend on the electrical properties of the solid grains. Although this occurs on the surface in a very thin layer, it may extend to considerable depths in dry or semiarid regions. Above the water table a mixture of air and water occurs, which

acts almost like a saturated condition. Porosity, the second factor influencing the electrical properties, is a measure of the relative quantity of pore filling. The porosity of most clays is high—about 40–60 per cent—compared to the 25–40 per cent porosity of sands and gravels. The permeability of most natural clays, however, is exceedingly low compared to that of sand and gravel. The ratio is approximately 1:10,000. These conditions act to control the electrolytic quality of the pore filling, because the alluvial sands and gravels of the humid region consist preponderantly of quartz particles and other only slightly soluble silicates. The clays, however, contain more minerals of higher solubility which are, in addition, in a finely divided form. In the sands and gravels the water tends to move freely over the relatively insoluble grains. In the clays the water is trapped within a mass of very fine, irregular, and more soluble particles. This prolonged contact with these finely divided minerals tends to bring more of them into solution and obviously increases the electrical conductivity in the clays over that of the less porous material. It is apparent, therefore, that the electrical-resistivity values of the fine-grained materials will be low, while the values for the sands and gravels will be high.

Resistivity instrumentation requires a battery, a potentiometer, a milliammeter, and four electrodes. In field practice the electrodes are set in a straight line and are equally spaced. The electrode intervals are expanded outward and the apparent resistivity observed for each interval. Roughly, the depth of current penetration is equal to the electrode interval. The observed values are plotted against the depth, and interpretation of the resultant curve is made. This is done by mathematical analysis, by matching against theoretical curves, or by matching against observed curves of known conditions. This technique is called depth profiling. When the electrode interval is kept constant and the whole system moved, the technique is called traverse profiling.

Choice of Methods

In general, those are the principles and field techniques of those geophysical methods currently applicable to ground water exploration. They are, however, only indirect methods, and water cannot be directly indicated by them, but they are aids and supplements to the understanding and the interpretation of the geology of the area studied.

Beside the limiting factors of techniques, the choice of methods to be used is also dictated by the conditions in which ground water commonly occurs and accumulates. These are:

1. Watercourses, consisting of a perennial stream together with the inclosing and underlying alluvium saturated with water that comes from the stream, from infiltration at the surface, or from adjacent water-bearing deposits

2. Loose water-bearing materials, chiefly sand and gravel of glacial drift and outwash deposits

3. Consolidated water-bearing rocks, of which limestone, basalt, and sandstone are the most important.

For the first two conditions, electrical-resistivity methods are most applicable, because they are rapid and have given good information, provided, of course, that sufficient differences exist in the electrical properties of the confining materials. For a buried valley, a series of depth profiles at closely spaced stations along a line crossing the valley would be established. Addi-

tional profiles are made so that the configuration of the bedrock surface and the lateral and vertical extent of the sand and gravel layers may be determined. For sand and gravel within glacial drift and glacial outwash and where a large area is to be investigated, a traverse profile would first be made around the area. Next, grid lines would be established across the area and more traverse profiles made along these lines. On the basis of the resultant observations, isoresistivity lines could be drawn, just as contours are, and the areas of high resistivity would then be investigated again with another series of depth profiles. This would determine the lateral and vertical extent of the high-resistivity materials.

The seismic methods would be employed much the same way except that a continuous profile around the area would not be made, mainly because the resistivity methods require, for best interpretation, an extension of the survey over a considerable distance, so that the rate of change of values can be measured and a background established. Seismic methods give a depth determination over the point of observation, whereas the resistivity values are an average over the electrode spacing. Seismic methods are subject, however, to more strict limitations in application: for example, in the deeply alluviated lower Mississippi Valley, they are only partially successful. Here, alluvial sands and gravels are underlain by fine-grained marine clays, and resolution of depth to this stiff clay is indeterminate because these clays transmit the elastic wave at a lower velocity than the water-filled sands and gravels.

In White County, Ill., an electrical-resistivity survey was made for the purpose of locating a source of water for injection into an oil-producing stratum and waterflooding it, in order to increase the amount of recoverable oil. In this area the deep bedrock valley which was formerly occupied by the ancient Wabash River is about 6 mi wide and is filled to an average depth of 125 ft. The upper 50–60 ft of the filling is silt and clay. Below this are extensive sand and gravel layers. There depth profiles were made along a few widely spaced lines crossing the valley. Other depth profiles placed along short cross-valley lines detailed the area of high resistivity. Borings, made for control and correlation of the resistivity observations, showed 50–70 ft of gravel and sandy gravel. These gravels were hydraulically connected with other gravels over a considerable distance in this buried valley.

Although resistivity methods have been best applied to unconsolidated materials, they have been employed with a degree of success in locating sinkholes and caverns in a limestone under a relatively thin cover. These were engineering studies for foundation purposes, but conceivably they could be used to locate solution cavities and fissures, which sometimes carry large quantities of water. This water is often susceptible to contamination, however, and the fissure system has recharge problems.

In the consolidated water-bearing rocks, neither resistivity nor seismic methods are of much direct value. Their chief value is that they can help in the solution of geologic structure problems.

Responsibility of the Consulting Engineer in Water Plant Construction

Murray A. Wilson

A paper presented on April 11, 1957, at the Kansas Section Meeting, Wichita, Kan., by Murray A. Wilson, Cons. Engr., Salina, Kan.

DISCUSSION of the subject of engineering responsibility could be limited to the periods of during and after construction, but the subject can be discussed more adequately and understandably by looking at a job as a whole and considering the engineer's responsibility all the way through.

Thinking is, in itself, a difficult occupation, and thinking in the abstract is the most difficult. Therefore, the best approach to this subject is to assume that there is a city of 2,000 population, which has decided that it probably needs a new water purification plant; it has found, perhaps somewhat to its astonishment, that it cannot order one from a mail order catalog; and that it will probably need to retain an engineer to help get the plant through its many stages of construction and into operation.

Responsibility of Planning

The engineer's responsibility begins when he first interviews the members of the governing body and appears before that group with the intention of persuading them to contract with him for his services. His responsibility at this stage consists in telling the men of this town the truth, as he knows it: the truth about himself, his organization, his experiences, and his accomplishments. Members of the governing body have some responsibilities, too, because they should investigate the man who is offering his services, talk to some of his former clients, and assure themselves that his way of doing business will be acceptable to them and to the citizens whom they represent. It is the engineer's responsibility not only to tell them the truth about himself, but also to outline to them the steps which must be followed to carry out the project and make it clear to them the part of the work which they themselves must perform, as well as the part for which he will assume full responsibility.

When the engineer gets the assignment, he starts to prepare the preliminary report which will be used as a basis for an election to determine whether bonds will be issued to construct the plant. Here is where technical responsibility really begins. Too often in the past the tendency has been for the engineer to get the preliminary report done with a minimum of effort and little, if any, research. In some instances, especially when competition is keen, the procedure has been to place a ridiculously low price on the preliminary report and then, by cutting corners and eliminating any original research or original thinking, to try to produce the report for the amount of money which he named as the fee. For the preliminary report to be worthwhile, it requires all the essential in-

vestigation needed for the design of the plant; and engineers and water superintendents might well expend the effort to conduct a campaign of education to get members of the governing bodies thoroughly convinced of the truth of this, so they can be persuaded to spend enough money for the preliminary report to enable the engineer to do it properly.

As the engineer assembles the basic data for the design of the plant, he should make sure that the town is in financial condition to undertake the project. He should satisfy himself that the water supply which is to be treated in the plant is adequate. If it isn't, he should certainly discuss the implications with the governing body and explain fully the future troubles that will arise from the construction of a plant which is not supported by a dependable and adequate supply. He should study the history of the town as to its rate of growth, and study it for any peculiarities which might cause it to deviate from the average performance of towns in its class. His report should outline, in terms which the members of the governing body can understand, the basic reasons behind his choice of the component parts of the plant, the reasons for size, a description sufficient to make the preliminary plans understandable, and an estimate of cost which should be sufficient to build what he has described, together with the other expenses which the town will incur in handling the project. He should send this report to the members of the governing body and to the water superintendent, and after they have had time to read and study it, he should meet with them to answer their questions and explain any and all items which are not clear to them. At the same time he should transmit a copy to the state board of health for its review and comments. It is his duty to help the men who represent the town understand what they can expect from such a plant and how it can be fitted into their water works system. At this stage, the basic responsibility of the engineer is to answer five questions: [1] Is the plant adequate?; [2] Will it work?; [3] Can it be built for the estimated cost?; [4] Can the community legally afford it?; and [5] Does the governing body understand the situation? If all can be answered in the affirmative, then the engineer is justified in feeling that he has been aware of his responsibilities up to this point.

In preparing the detailed plans for the project, the engineer should review the problems pertaining to it, review the data which he has previously assembled, and fill in all the missing links by obtaining the additional facts he needs to complete the basic data. As the design proceeds, he usually finds that minor changes in his preliminary layout need to be made, and occasionally major changes seem desirable. He should bear in mind, however, that his client has approved a certain arrangement and general layout for the plant, and he should keep them informed of all but the most insignificant changes and get their approval. He should search diligently to find out what new equipment, methods of control, and materials of construction are available, and find out whether they have proved satisfactory under similar conditions and whether they would be applicable to his current project. He should see that the plans are prepared in sufficient detail that anyone familiar with water treatment could analyze the plant, and particularly the plans should be so arranged and in enough detail that a contractor may build the job without

guessing and with a minimum of confusion. His specifications should be written understandably, should contain definite instructions about everything he wants done, and be entirely free of ambiguous and extraneous phrases.

It is the engineer's duty, as soon as the plans and specifications are completed, to submit them to the board of health for review and approval. At the same time he should assist the city clerk or water superintendent in filling out the formal application which is the official request from the city to the board of health for permission to construct the proposed improvement. He should be present at a session with the governing body and explain the plans so that it will have a clear understanding of what the final results will be. If the city already has a water superintendent, the engineer should go still more into detail with him and be sure that he thoroughly understands the plans from beginning to end. If the city does not already have a water superintendent, it should acquire one at this stage of the project so that he may become familiar with all the details of design and construction as the work progresses.

In fact, throughout the preparation of the preliminary report and of the working drawings, the engineer should confer with the water superintendent, if there is one, not only for diplomatic reasons but also for the good of the project. More often than not the superintendent will have many pertinent and practical ideas which will improve the operation of the plant. Both engineer and superintendent, however, should approach the problem with an open mind, because even in this business it is very rare that one individual knows all the answers. They should be willing to compare and trade ideas from the standpoint of mutual helpfulness and not with the hope that one of them will finally be able to expose the ignorance of the other. Too often the designer has not had sufficient operating experience to foresee the difficulties which arise with most operations, and often he does not realize that some slight shifts in arrangement would materially reduce the effort required to keep the plant in operation. He should welcome the suggestions of a practical nature which will add to the comfort and ease of operation. On the other hand, he should be just as firm in vetoing impractical suggestions which are sometimes made by men who are not thoroughly familiar with the theory of water treatment, and should never lose sight of the fact that the design is his basic responsibility. He should never permit diplomatic or political considerations to allow him to accept faulty design.

Responsibility During Construction

Assuming that those problems have all been settled, the contract has been let, and the project is under construction, the engineer's responsibility now takes on a more concrete form. Generally speaking, it is his duty to see that the contractor builds the job exactly as shown on the plans and as described in the specifications. This is not to imply that the plans and specifications which form a part of the contract are related in any way to the laws of the Medes and Persians, "which altereth not." Since the engineer and his assistants are human beings, it is inevitable that errors and omissions will show up in the plans and specifications as these latter are being translated into concrete and steel. So on every job, there will be occasions when deviation

from the plans or specifications will be found desirable, and this should be accomplished by change orders submitted to and agreed to by the parties to the contract and made a part of the permanent records. He should require shop drawings to be submitted on everything that is put together away from the job site: such as reinforcing steel, equipment, and other prefabricated items. It is his duty to check these against the plans and specifications and against each other to make sure that the units will work together under any of the prescribed conditions. When the contractor reaches the subgrade for his footings, they should be examined to see if they are equal to the assumptions made during design, which were necessarily based on the data secured from test holes. If he finds a variation in the supporting power at any point, he should not hesitate to call for such additions as are necessary to make the structures sound and safe. Such additions of course call for a change order which carries with it additional pay to the contractor, and it is the engineer's responsibility to see that the price agreed upon is fair to both parties and that the governing body understands the necessity for the additional work. It is his duty to see that materials which are incorporated into the structure meet the quality as described by the specifications and that they are constructed and put together in a neat, workmanlike, and satisfactory manner. He should check reinforcing steel for size, spacing, and position, and should check all portions of the structure for dimensions and elevations. He should keep clearly in mind the uses to which the various structures will be put and insist on the requirements which special uses may make necessary. He should insist on neatness and good workmanship, universally, whether the results will be exposed to public view or not.

As equipment is delivered to the site, he should check it to see if it is of the size, capacity, and quality of that which was approved in reviewing the shop drawings. As this equipment is installed, he should keep constantly in mind what each piece of equipment will be doing and make sure that it is installed in such a way that it will operate. After the installation is complete, each piece of equipment should be operated to be sure that it actually performs the function for which it was ordered. The feed machines should be calibrated and operated under all conditions and rates to be sure that they cover the full range of specified delivery rates. The basins should be thoroughly tested, and any seepage or leaks should be repaired at once. Before the project is finally accepted, the engineer should require the contractor to put the plant through a complete operation, make sure that all the structures are usable, and that all the machines really work.

Responsibility After Construction

When the plant is complete, the final estimate to the contractor has been approved, and the state board of health has examined the plant and approved it, it is then ready for the city to take over and operate completely. It is also the responsibility of the engineer to make sure that the city's plant superintendent understands what he is about to do. In other words, he should know how the plant was built and how to operate it, and should have clearly in mind the limitations, both with respect to quantity and quality, which are in-

herent in the plant. The engineer should provide the superintendent with as-built drawings, which show how the plant was constructed and where the hidden pipelines are actually located. He should provide sufficient operating diagrams and operating instructions to enable the superintendent to refresh his memory from time to time until he becomes thoroughly familiar with all the details. He should provide the superintendent with a list of equipment showing the make, size, serial number, manufacturer, and the address of the nearest source of parts and repairs. He should call to the attention of the superintendent the critical points in the process within the plant and make sure that the superintendent understands the order in which valves and switches should be closed or opened, and in general, the sequence of any set of operations.

Responsibility of City

As in any dealings between human beings, responsibility does not always rest wholly on one side, and the relationship of a consulting engineer to the governing body of the city is no exception. There are some responsibilities which the governing body has toward the consulting engineer, and these should be pointed out along with the responsibility of the engineer. The city should not expect the consulting engineer to take a green hand, altogether unfamiliar with the water plant operation, and to train him for the job of superintendent at his own expense and without cost to the city. The engineer should see that the governing body understands clearly the nature of the plant and the type of operation which they can expect from it. On the other hand, the governing body should not criticize and belabor the engineer for the fact that the plant is not fully automatic, that it is not self-operating, and that care and operation of the plant requires labor and intelligent management. Putting it another way, the governing body has no right to blame the engineer for the fact that it has a pickup truck instead of a Cadillac, when the pickup truck was all that it could afford and the engineer clearly explained that such was the type of equipment it was going to be furnished. Situations of this nature have developed into a three-way battle which ended with everybody concerned a loser.

The proper conduct and procedures of the consulting engineer are clearly and concisely set forth in the codes of ethics of any of the reputable engineering societies. An engineer who adheres to these principles will certainly avoid any justifiable criticism. The legal liability of the engineer is not so clearly determined. To operate legally, the consulting engineer must, of course, be licensed to practice in the state in which he does business. A few courts have decided that a man could not collect an engineering fee if he was not properly licensed as a professional engineer. There have also been several interesting cases decided in various states of the union in which the court has ruled that the engineer or architect involved in the project could not collect his fee if the project failed to be constructed, due to the fact that bids could not be obtained within the engineer's estimate. One occasionally hears of an argument developing between the engineer and his client which seems to be based on political reasons where some member of the governing

body or the superintendent attempts to use the engineer as a scapegoat by blaming him for all the points in which the plant has fallen short of absolute perfection, many of which are entirely outside the control of the engineer. This situation sometimes develops, due to the lack of communication and the failure of the engineer to explain properly and clearly the details of the plant to those who rightfully merit such explanation. There are occasions, however, when it is purely a clash of personalities or an attempt of certain individuals to cover up their own shortcomings and mistakes by blaming their troubles on the engineer.

The whole matter can be summed up briefly by pointing out that the only way that a job can be done smoothly and satisfactorily is for the governing body to select an engineer in whom it has confidence, and for the engineer to deal honestly and openly with the members of the governing body, leading them to a compromise between what they think they want and what they can really afford, and then making sure that they get exactly what was specified and designed. If each of the many individuals involved in a project of this kind will honestly try to understand the other man's viewpoints, abilities, and limitations, the work can go along on the basis of mutual trust, which in the end will provide the best job and the most satisfaction for all concerned.

Effects of Seeding Cumulus Clouds

Roscoe R. Braham Jr. and Louis J. Battan

A contribution to the Journal by Roscoe R. Braham Jr. and Louis J. Battan, both of the Cloud Physics Laboratory, Dept. of Meteorology, Univ. of Chicago, Chicago, Ill.

THE clouds of the atmosphere are a natural resource, with the same potentialities as the forests, fields, lands, and streams. The desirability of finding methods for using this resource is uppermost in the minds of those who have studied the problem of satisfying the world's insatiable appetite for water. It is understandable, therefore, that cloud seeding has been considered by some as a last hope for a dwindling water supply and by others as a form of insurance for keeping water supplies at comfortable levels.

Cloud seeding, unfortunately, has been the subject of controversy since the initial experiments more than 10 years ago. The roots of the controversy lie in an inadequate knowledge of the physics of natural and artificially induced precipitation and in doubts about the suitability of methods for assessing the results of most past cloud-seeding experiments.

Seeding Research at Chicago

In the spring of 1952, scientists of the Cloud Physics Laboratory of the University of Chicago Department of Meteorology undertook a systematic study of the effects of seeding cumulus clouds. This study was begun because of the huge gap between the reports of the commercial cloud seeders and the findings of the US Weather Bureau's cloud physics project (*1*). This research, begun in 1952, has continued to the present time.

The objective of the cloud physics research at the University of Chicago is the formulation of a consistent and immediately applicable picture of the natural processes of cloud and precipitation development with a view toward testing the possibility that one can modify these processes and influence the natural behavior of clouds. Emphasis is divided between conducting carefully controlled seeding experiments and carrying out studies of the fundamental physics of natural precipitation. Chicago scientists believe that the greatest likelihood of success in cloud seeding will come from basing seeding technology upon a firm foundation of fundamental understanding of natural cloud and precipitation mechanisms. By carrying out a large number of experiments—identical in every respect except that the clouds in half of the experiments are treated whereas those in the other half are not—the experimenter is in a position to learn about natural processes from the nontreating experiments and to determine the effect of seeding by contrasting the treating and nontreating experiments. Such a scheme is used to permit the investigators to draw valid conclusions about the effects of the seeding in the face of large natural variations in the "raininess" of clouds.

Engineers who are pressed to meet rising demands for water in the face of falling water supplies may find it difficult to accept a plan where only half the seeding opportunities are used. It is a hard fact, however, that there is no other technique available today for carrying out an experiment suitable to clear-cut analysis and statement of results when dealing with weather phenomena which have large natural variations.

Emphasis on Cumuli

One of the first steps in solving a difficult problem is to break it up into a series of smaller ones which are individually less difficult. The Chicago cloud physics group undertook a study of the effects of seeding convective clouds. Clouds of this type—cumuli, for example, commonly seen in thunderstorms—provide substantially all of the summer precipitation in the United States.

Cumulus clouds are particularly suited to systematic cloud-seeding studies because they are individually identifiable and retain their identity throughout their histories. The research reported in this article was carried out from instrumented airplanes which were used to place the seeding material directly in the active regions of the clouds and to follow the development of precipitation regions within the clouds. In this way it was possible to strip the problem of the difficulty of knowing where and when the seeding materials reach the clouds—as is invariably the case with seeding from ground generators.

Airplanes—three B-17's and one B-26—for use on this research, and financial support for the investigations were supplied by the US Air Force under contract between the University and the Air Force Cambridge Research Center. During the period from August 1953 through September 1956, field operations were carried out from Chanute Air Force Base, Rantoul, Ill.; Ramey Air Force Base, Puerto Rico; and Davis-Monthan Air Force Base, Tucson, Ariz. In all, 247 research flights were conducted. On these flights, a total of 184 large, bulging cumulus clouds were treated and followed throughout their life span in order to establish the behavior of the cloud following treatment. Over 200 cumuli were followed in detail through their natural life and well over 1,000 similar clouds were studied in order to determine their "rainability index"—the probability of rain formation as a function of measurable physical qualities of the clouds. This report is based upon the analysis of these data.

Experimental Program

In preparing a cloud-seeding experiment, one must keep in mind the basic problem of separating the effects of seeding from purely natural effects. This requires special precautions against subjectivity and bias of the experimenter and care to insure that natural meteorological variations will not mask the effects from seeding. In order to accomplish these objectives a system was set up whereby the scientist selecting the clouds for study had no knowledge of which clouds were actually seeded. On each day at a particular location, clouds were worked in pairs—one seeded and one unseeded—so that the behavior of the seeded cloud could be assessed against that of the unseeded.

In designing the experiment it was decided to study the initiation of precipitation divorced, if possible, from the problem of producing more rainfall.

By the term "initiating precipitation" is meant the process of causing precipitation to form in clouds in which it would not form through natural causes or, alternatively, causing it to occur significantly earlier than it would have occurred naturally. These processes are to be contrasted to "increasing the rainfall" which refers to the act of starting *any* processes which will result in greater amounts of rain at the ground than would have occurred through natural causes. In all likelihood, to be successful at initiating rain will automatically mean increasing rain at the ground, and the latter must obviously be the ultimate goal of cloud seeding.

The distinction between these two mechanisms is made because the ability to measure the results is totally different in the different situations. Rainfall at the ground is measured by means of surface gages which are small and, usually, widely spaced compared with the dimensions of individual rain cells. This means that it is difficult to determine the amount of rain falling from a single storm or group of storms. The problem is particularly acute in the case of the convective storms which characterize summer rainfall. By taking averages over long periods of time one can determine the rain over an area from rain gages. Unfortunately this approach is not satisfactory for assessment of seeding trials because it increases the likelihood of error arising from temporal changes in natural rain in direct proportion to the length of period considered. Radar can be used to detect the presence of rain and present research offers promise of making radar useful for measuring the quantity of rain. This system, however, is still in its infancy.

Contrasted with the problems of measuring rain amounts at the surface, perfectly acceptable means are available for measuring the initiation of precipitation within any given cloud. Properly operated radar can provide a "yes" or "no" answer to the question of rain initiation. This objective capability of the radar was one of the decisive factors in the decision by the university to center its investigation about "precipitation initiation" rather than "precipitation increase."

Flight Program

To carry out the research it was necessary to make a series of measurements of relevant physical parameters of a large number of seeded and unseeded clouds. Three B-17's were used to carry personnel and the equipment required to [1] free-air temperature; [2] dew point temperature; [3] cloud liquid-water content; [4] cloud droplet size and number; [5] icing rate; [6] electric fields within the clouds; [7] precipitation (time of formation, location, and size of precipitation area); [8] vertical air motions; and [9] atmospheric particulates which could serve to nucleate cloud droplets.

Additionally, the airplanes were equipped to dispense seeding reagents. Two such reagents were used. Some of the clouds were treated with dry-ice pellets released within the clouds at a rate of 50 lb/mi of flight. Others were treated with a spray of water droplets released from a large tank within the airplane at the rate of 450 gal/mi of flight.

The methods used for selecting the individual clouds for investigation are thought to be novel in cloud-seeding trials. Since the primary objective was to determine the microphysical conditions associated with natural rain *initi-*

ation and to attempt to create precipitation in clouds in which it would not occur naturally, it was essential that the cloud under study contain no precipitation *before* study of it began. Thus the first step in the study was for the senior investigator to select a cloud which was suitable for the experiments at hand and which he thought likely to produce rain through natural causes. To be suitable for dry-ice seeding, the clouds had to be firm in appearance, have a solid base, appear to be growing (signifying a good updraft), extend to a height where dry-ice seeding could be accomplished all that seeding could hope to do. When it had been determined that the cloud did not contain precipitation, the senior investigator declared the cloud to be suitable for continued study.

The method of studying the clouds consisted of making repeated passes through the cloud to permit repeated measurements of all the cloud parameters. The first pass through the cloud was called "the seeding pass" although only half of the suitable clouds were seeded. After the senior investigator had declared a cloud suitable, another,

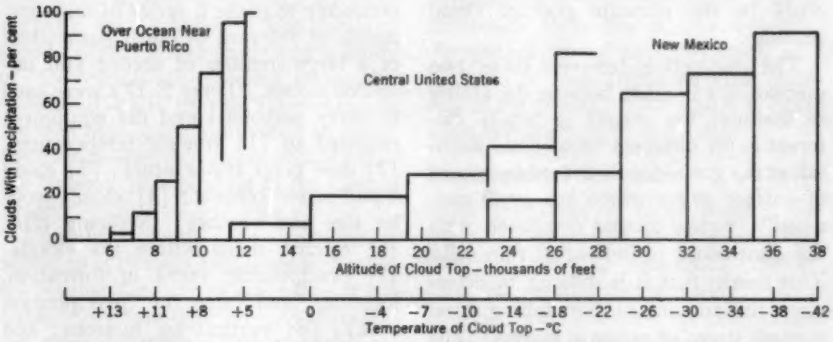
Clouds With Precipitation – per cent
100
80
60
40
20
0
Over Ocean Near Puerto Rico
Central United States
New Mexico
6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
Altitude of Cloud Top – thousands of feet
+13 +11 +8 +5 0 −4 −7 −10 −14 −18 −22 −26 −30 −34 −38 −42
Temperature of Cloud Top – °C

Fig. 1. Relation of Precipitation Development to Cloud Height

Census was taken of cumulus clouds in three geographically different regions indicated.

effective (average temperature of tops of clouds at times of seeding was −7°C), and had to be located where natural seeding (drizzle, falling ice crystals, and so on) was unlikely.

Once a cloud had been selected for study it was examined for the presence of precipitation. A special radar mounted on the nose of the airplane was used for this purpose. If precipitation was found to exist inside the cloud (even though it did not appear at the cloud base) the cloud was abandoned), for in this event, nature had within the airplane, opened a set of sealed instructions which determined whether or not a particular cloud was to be seeded. The actions of this second investigator were unobserved by the first who had no way of determining whether or not the seeding material had been released from the airplane.

Regardless of whether or not the cloud was seeded, the airplane made repeated passes through it, until the cloud developed into a large, vigorous rainstorm or until it dissipated through evaporation. An average of 30–45 min

was required to complete the measurements on a single cloud. A second cloud was then selected and the entire routine repeated except for the seeding, for clouds were always studied in pairs, and if the sealed instructions directed that the first cloud be seeded, the second cloud was not—or the reverse. This flight procedure continued until all suitable clouds in the area had dissipated or until the flight time of the airplane was exhausted. Studying the clouds in pairs permitted a comparison between seeded and unseeded clouds on a given day. In this way the day-to-day variations in the probability of formation of the clouds could not confuse the seeding results.

Cloud Census

Detailed studies of the kind described above were time consuming although indispensable. It was imperative, therefore, that a corollary study be made to relate the probability of natural rain in a given cloud to the more tractable physical parameters of the clouds. Such a study—called a cloud census—requires examination of a large number of clouds in a short time, in order to determine which have developed rain through natural processes. The results of such studies are shown in Fig. 1.

Data of this kind are very revealing. For example, it is shown that natural precipitation develops in every cumulus cloud in the Caribbean *before* it grows to a height where ice crystals could possibly be involved. Seeding such clouds with ice-forming material such as dry ice or silver iodide could not possibly initiate precipitation. For this reason, cloud seeding in the Caribbean was done with a spray of water droplets.

The cloud census of cumulus clouds in central United States and the Southwest shows that it is rather unlikely that precipitation will develop before the time the clouds reach the freezing level. On the basis of these data only, one might conclude that there is a possibility for rain initiation through dry-ice or silver iodide seeding. Other sources of data not reported here lead one to question the magnitude of this possibility, however (*2*).

Water Spray Seeding

Because of the importance of non-ice mechanisms in rain formation in many areas of the world, a large part of the study was devoted to seeding cumuli with water spray. In this effort, the water was released from a 400-gal tank in the bomb bay of the airplane. In early studies, the water was dumped through a 4-in. diameter quick-release valve which provided a flow rate of about 130 gal/mi of flight. After sev-

TABLE 1

Results of Water Spray Treatment of Tropical Cumuli

No. of Pairs of Clouds	Rain(+) or No Rain(−)	
	Treated	Untreated
With Small Valve		
3	+	+
3	+	−
4	−	+
22	−	−
With Large Valve		
5	+	+
6	−	+
17	+	−
18	−	−

eral experiments which failed to initiate rain, the valve was changed to a 10-in. square dump valve which increased the flow rate to about 450 gal/mi. This larger flow rate proved effective in rain initiation.

The results of the water spray treatment experiments are shown in Table 1 where it can be seen, for example, that there were four pairs of clouds in which the untreated produced rain (as revealed by a radar echo) and the treated clouds produced none.

The large-valve data are statistically quite significant. The probability that one could get the observed 17–6 split had there been *no* effect from the seeding is 0.017. In other words, the experiments show, with high statistical significance, that the release of water at the rate of 450 gal/mi into cumulus clouds initiated precipitation in clouds where it would not have occurred naturally.

The data from the small-valve studies do not permit the conclusion that the spray had an effect on the development of precipitation. Further engineering of the spray release might improve the effectiveness of the spray and produce positive results from flow rates less than the 450 gal/mi of the large valve.

This same positive effect was apparent when the other physical parameters of the clouds were studied. For example, in the seeded clouds, the rain formed earlier than in the unseeded ones. A study of the rain echoes from the seeded clouds showed their size and duration to be essentially similar to those of the unseeded clouds. Thus, these studies show that one *can* initiate rain in cumuli through water seeding and that the rain region thus formed will be similar to naturally formed ones. One must conclude, therefore, that water spray increases the water falling to the ground from cumulus clouds. It must be borne in mind, however, that clouds suitable for water seeding are rather small—less than 8,000 ft thick—and that a substantial flow of water spray is required. Larger clouds rain naturally and smaller flow rates appear to be ineffective. It should also be borne in mind that the bulk of these data were taken in the Caribbean area. It is likely that the larger nonraining cumuli of the continent would yield proportionately more water through water-spray methods.

Seeding With Dry Ice

Dry-ice treatment studies have been carried out in central United States and in the Southwest. Unfortunately, it was necessary to terminate this phase of the research before adequate data were obtained; nevertheless, the data which were obtained are suggestive. The results of the dry-ice treatment tests are shown in Table 2. It can be seen that of the 31 pairs of dry-ice treated clouds, 16 pairs contribute information concerning the efficacy of dry-ice treatment. These 16 pairs are split 9–7 in favor of positive results. The probability of obtaining a 9–7 split in a sample of 16, should there be no

TABLE 2

Results of Dry-Ice Treatment of Subcooled Cumuli in the United States

No. of Pairs of Clouds	Rain(+) or No Rain(−)	
	Treated	Untreated
1	+	+
7	−	+
9	+	−
14	−	−

effect, is rather high and it must be concluded that these neither support nor deny the hypothesis of positive seeding effects.

The data do show quite conclusively, however, that dry-ice seeding of cumulus clouds will not lead to the spectacular effects that have sometimes been described as occurring following single seeding trials. It must be concluded that it is likely that such seemingly extraordinary developments of clouds following seeding with dry ice are due solely to natural meteorological processes.

The inconclusive results following the seeding of cold, supercooled cumulus clouds with dry-ice pellets have been the subject of a great deal of study. At first thought these results would appear to be incompatible with laboratory findings. There is no question but that dry ice can be used to cause a supercooled cloud to convert to ice crystals under laboratory conditions. Why did the clouds in nature not show similar effects?

The possibility of "overseeding" * was first considered; however, the design of the research plan meant that a complete record of the form and size of the cloud particles was available for the entire life of the storm. The collections of cloud particles showed that liquid water particles were present throughout the life of the cloud, even though the clouds had been seeded. In other words, the ice particles were not starved for water and clouds had not been overseeded.

The most likely reason for the failure of dry-ice seeding to produce positive effects is that the average life of a cumulus cloud, following first inspection, is approximately the same as the time required for small ice crystals to grow into particles large enough to be important in the development of precipitation. The probability that natural rain will develop in a cloud is directly proportional to the length of life of the cloud. In the University of Chicago studies, every unseeded cumulus cloud lasting more than 25 min developed precipitation through *natural causes*. The average life of a cumulus cloud after seeding was between 15 and 20 min. About 15 min is required for dry-ice (or silver iodide) seeding to produce precipitation of particles, *after such particles have begun growing within the clouds*. It follows, therefore, that the average cloud was beginning to dissipate by the time the precipitation particles grew to detectable sizes.

Summary

Through a long and very careful series of field experiments, it has been shown that the formation of precipitation can be induced in relatively small cumulus clouds, in which it would not form naturally, through the use of water spray released inside the clouds.

Experiments involving the treatment of supercooled cumulus clouds with dry ice released into the cloud at levels where the temperatures were favorable for ice crystal growth did not result in demonstrable seeding effects—either positive or negative. It was shown that it is unlikely that dry-ice seeding

* The term overseeding is used to denote the condition where the number of ice-forming nuclei is too large for the amount of liquid water in the cloud. If overseeding were to occur, there would be insufficient water present to permit the individual ice crystals to grow large enough to become effective in the precipitation process. Research at the University of Chicago suggests that it is impossible to overseed the atmosphere with any reasonable number of existing seeding generators.

will result in spectacular development of cumulus clouds for spectacular increases in the amount of precipitation from them, or both. The reason for this is thought to be that the lifetime of an average cumulus cloud is about equal to the time required for introduced ice crystals to grow into precipitation particles. This does not mean that seeding with ice-forming materials cannot have positive effects. Successful rain initiation conceivably can result from: [1] the initiation of precipitation in marginally small clouds which would not have lasted long enough to rain naturally; [2] the growth of small (nonprecipitation) ice crystals in one cloud and the transfer of these particles to a second cloud as the first cloud dissipates; or [3] resupplying clouds with ice nuclei after the natural ones have been consumed in the precipitation processes. Only further basic research can solve these problems.

Acknowledgment

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Improved Membrane Filter Medium for the Detection of Coliform Organisms

Charles W. Fifield and Charles P. Schaufus

A contribution to the Journal by Charles W. Fifield, Research Assoc., Dept. of Food Technology, Massachusetts Institute of Technology, Cambridge, Mass., and Charles P. Schaufus, Chief, Bacteriology Lab., Millipore Filter Corp., Watertown, Mass.

AN outstanding advance in methods applied in the field of water bacteriology during the past several years has been the development of the membrane filter technique. Of the many advantages of the technique, perhaps the most significant is the ability to concentrate bacterial organisms from very much larger and, therefore, more representative samples. The successful recovery of distinguishable colonies depends to a large extent, however, on the adaptation of a suitable medium.

In 1951, Clark, Geldreich, Jeter, and Kabler (*1*) advocated a two-step procedure using Albimi M * medium and a modified Endo medium (EHC—USPHS Environmental Health Center) for use in the examination of water for the presence of coliform bacteria. Later, Goetz and Tsuneishi (*2*) suggested the use of special indicator broth media. In 1953, Taylor, Burman, and Oliver (*3*) modified MacConkey's medium for use with the membrane filter. More recently, Hajna and Damon (*4*) reported the adoption of buffered desoxycholate lactose broth as a one-step medium for the detection of the members of the coliform group.

Of the coliform media suggested, perhaps the most widely used today are the EHC two-step procedure and M-Hajna Damon (M-HD) Endo broth (*5*); both have decided disadvantages. Because of its two-step nature (requiring an incubation period on enrichment medium prior to incubation on a selective medium), the EHC procedure is time consuming. The coliform recovery, however, is good. The M-HD Endo medium lacks clear-cut sheen differentiation of many of the intermediate coliform colonies.

Recently, an improved medium, MF-Endo,† was placed on the market. Preliminary tests showed the medium to be superior in several respects to existing coliform media.

This investigation was conducted to determine the recovery and selectivity of this new medium for members of the coliform group. Confirmatory tests, carried out on the sheen and nonsheen colonies produced, involved the inoculation of lactose broth and nutrient agar slants for microscopic observation in accordance with *Standard Methods* (*6*, pp. 395–404).

A coliform organism count comparison was also made between MF-Endo

* A product of Albimi Labs Inc., Brooklyn, N.Y.

† Developed by the Millipore Filter Corp., Watertown, Mass., and available through Baltimore Biological Laboratory, Baltimore, Md., Difco Laboratories, Inc., Detroit, Mich., and Albimi Labs, Inc.

TABLE 1

Formulation of MF-Endo Medium

Ingredient*	Amount g/l
Tryptose	10.000
Thiopeptone	5.000
Casitone	5.000
Yeast extract	1.500
Lactose	12.500
Sodium chloride	5.000
Dipotassium phosphate	4.375
Monopotassium phosphate	1.375
Sodium lauryl sulfate	0.050
Sodium desoxycholate	0.100
Sodium sulfite	2.100
Basic fuchsin	1.050

* Ingredients are added to distilled water to which 10 ml/l of 95 per cent ethanol has been added. The medium is mixed and heated to boiling, but not autoclaved.

and M-HD broth using three different natural water sources. Further, a pure-culture study was undertaken to determine the recovery of *Esch. coli, Aer. aerogenes,* and five coliform intermediates.

Procedure

MF-Endo medium was made up according to the formulation given in Table 1. M-HD Endo broth was made up from a dehydrated medium. When not in use, the formulated medium was stored in a refrigerator and was not used after more than 4 days.

Before the filtration procedure, 2.0 ml of the medium was placed on an absorbent pad in a disposable plastic petri dish. Membrane filters,* packed for autoclaving and sterilized at 121°C for 10 min, were employed. After filtration of the sample, the funnel was rinsed with about 25 ml of sterile water. The filter was removed and placed on the nutrient pad in the plastic petri dish. The dish was inverted and incubated at 37°C for 18 hr.

Fifty-one samples, from three sampling sites, were analyzed. Each sample was filtered in quintuplicate and, from each of the 255 filters, five sheen and five nonsheen colonies were picked and seeded in lactose broth tubes. At the end of 48 hr, all lactose broth tubes showing gas were streaked on nutrient agar slants and were examined microscopically 24 hr later. Gram-negative nonspore-forming rods, positive for gas in lactose, were recorded as coliform organisms.

* Millipore filters, HA 47 mm; a product of Millipore Filter Corp., Watertown, Mass.

TABLE 2

*Confirmation of Sheen and Nonsheen Colonies Grown on MF-Endo Medium**

Sampling Point	Sheen Colonies†				Nonsheen Colonies‡			
	Confirmed		False Positives		Confirmed		False Negatives	
	No.	Per Cent	No.	Per Cent	No.	Per Cent	No.	Per Cent
Charles River at Cambridge	430	86.4	75	13.6	504	96.7	17	3.3
Charles River at Watertown	457	96.0	19	4.0	400	88.7	51	11.3
Fresh Pond at Cambridge	249	99.6	1	0.4	240	96.0	10	4.0
Summary	1,136	92.3	95	7.7	1,144	93.6	78	6.4

* From 51 samples, or 255 filters.
† Total of 1,231 colonies.
‡ Total of 1,222 colonies.

It was necessary to pick all colonies while the filters were moist. The filters were then removed from the nutrient pads and dried for 1 hr on clean blotting paper, after which the sheen colonies were counted. The dried filter was placed between two pieces of glass and viewed under 16× magnification with direct lighting.

Nutrient agar plates, used as controls in the pure culture studies, were incubated at 37°C for 24 hr.

Discussion

It is imperative that any medium used for water quality analysis should exhibit a minimum of false positives and false negatives. The results of the confirmation of 1,231 sheen colonies and 1,222 nonsheen colonies are cited in Table 2. These results show a low percentage of false positives (7.7 per cent) and false negatives (6.4 per cent). Furthermore, this work was carried out in two separate laboratories, with comparable results.

The coliform density at one of the Cambridge sampling points was relatively low, and occasionally colonies were picked which, when wet, appeared to show a characteristic sheen but which would not have been reported as sheen colonies when dried. Although this error resulted in a higher number of apparent false positives, it was not considered in the tabulation of the data.

As well as being selective, a good differential medium must also show a high rate of recovery of the desired organisms. A coliform count comparison on 21 samples was made between MF-Endo and M-HD broth. The count given for each sample (Table 3) represents the average of five filters. Only definite sheen colonies were recorded.

The difference in the quality and intensity of the sheen of colonies grown on the two media was immediately apparent. The MF-Endo medium removes most of the uncertainty of whether or not a colony has sheen. This fact, along with the increase in coliform recovery of approximately 35 per cent (Table 3) for these particular sampling points, indicates a distinctly superior medium.

TABLE 3

Comparison of Sheen Counts on MF-Endo and HD Endo Media*

Sample†	MF-Endo Count	HD-Endo Count
1	15.8	12.6
2	21.2	14.8
3	9.0	4.0
4	216.0	108.6
5	195.4	127.0
6	181.6	109.8
7	22.6	19.2
8	30.4	25.0
9	21.0	11.2
10	19.4	6.6
11	69.4	52.2
12	14.4	8.8
13	257.8	210.8
14	14.2	6.4
15	9.4	2.2
16	28.0	20.3
17	67.0	45.7
18	12.0	10.4
19	24.8	18.4
20	25.6	18.4
21	56.2	42.8
Totals	1,311.2	875.2
Per cent	100‡	66.7

* Each figure represents the arithmetic average of the coliform count on five replicate membrane filters.

† Samples 1–15 were taken from the Charles River at Cambridge; samples 16–21 from the Charles River at Watertown, Mass.

‡ Regarded as 100 per cent recovery for purposes of comparison.

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Since the entire coliform group of organisms, and not *Esch. coli* alone, has sanitary significance, a pure culture study was undertaken to determine the

TABLE 4

Recovery of Coliform Organisms on MF-Endo Medium Using Controls

	Strain						
	W-46	W-2	W-4	W-35	W-1	W-5	I-58
IMViC*	−++−	++−+	−+−−	++++	++−−	−+−−	−−++
Sheen on MF-Endo†	20.8	27.6	47.6	25.4	61.4	50.4	44.6
Count on nutrient agar†‡	21.5	28.8	52.8	26.6	59.5	51.2	50.2
Recovery—*per cent*	97	96	90	96	103	99	89

* IMViC refers to the combination of four tests for biochemical reactions of a coliform culutre, expressed by plus and minus signs. In proper order, these are the indole, the methyl red, the Voges-Proskauer, and the citrate tests (*6*, pp. 390–91).

† Average of five replicate filters and plates for each coliform type.

‡ Used as control.

recovery rate of *Esch. coli, Aer. aerogenes,* and five coliform intermediates. In all cases, sheen counts on MF-Endo were compared with total counts on nutrient agar plates. The results given in Table 4 show excellent recovery rates for all cultures used.

Summary

1. An improved medium, MF-Endo, was investigated for use in enumerating members of the coliform group of organisms on membrane filters.

2. The medium produces colonies with excellent, clear-cut sheen characteristics.

3. Of 1,231 sheen colonies, picked from 255 filters, 92.3 per cent were confirmed as belonging to the coliform group. Of 1,222 nonsheen colonies, picked from 255 filters, 93.6 per cent were confirmed as not belonging to the coliform group.

4. The coliform recovery rate of MF-Endo was found to be significantly higher than the recovery rate of M-HD Endo broth.

5. Pure culture studies indicated a high rate of recovery for *Esch. coli, Aer. aerogenes,* and five coliform intermediates.

Acknowledgment

The authors wish to express their appreciation to Marcia Tully for technical assistance with the laboratory procedure and to Harold Thomas, Department of Sanitary Engineering, Harvard University, for his assistance in the interpretation of the data.

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Growth of a Water System

Walter M. Franklin

A paper presented on Nov. 12, 1956, at the North Carolina Section Meeting, Charlotte, N.C., by Walter M. Franklin, Gen. Supt., Charlotte Water Dept., Charlotte, N.C.

THE first water system was placed in operation in Charlotte, N.C., in 1881. Records indicate that prior to that time the closest approach to a public water supply in the city was four cisterns, each one located in one of the four wards of the city. These cisterns were used exclusively for fire protection. Apparently water for domestic consumption was obtained from numerous private wells. In 1881, the city council entered into a contract with a private corporation to construct a water supply and distribution system to serve the entire city. This company was named the Charlotte Water Works Co. The terms of the contract with the company stated that it was to supply adequate water for the needs of the city's population, both for fire protection and domestic use. The quantity of supply indicated that no less than 100 gpcd would be made available. The city agreed to pay $2,000 annually for 45 fire hydrants, with additional hydrants billed at $50 each. The source of supply was Sugaw and Briar Creeks, two of the three streams which flow through the city limits.

Apparently the people were not too happy with the service provided by this company. The records indicate that numerous discussions were held in the city council meetings regarding the inadequate service being furnished by the company. Therefore, in 1899, the city purchased the interest of the private water company for the sum of $226,000. This money was made available by bond election after the proper legislative approval had been given the city. In 1905, a 5-mgd capacity filter plant and pumping station was placed in operation on Irwin Creek at a point about ¾ mi east of the present Vest Station, and the Briar and Sugaw Creeks source of supply was abandoned. The Sugaw Creek drainage area was abandoned in order to allow the construction of a large textile manufacturing plant.

Early Development

In 1911, the Irwin Creek supply failed to the extent that a severe water shortage was experienced. Water was brought into the city by rail from nearby sources of supply. The city was then forced to utilize the Catawba River as a source. Consequently, a pumping station was built on this river at a site a few hundred yards north of the present pumping station, and a 24-in. cast-iron pipeline was laid from the river to the filter plant on Irwin Creek. A raw-water storage reservoir, known as the Hoskins Reservoir, having a capacity of approximately 60 mil gal at the outset, was built on the highest ground between the river and

the filter plant, so that water would flow by gravity from the reservoir to the filter plant. Both steam and electricity were used for distribution system pressure.

In 1920, the Duke Power Company built what is now known as the Mountain Island Lake, this impounding reservoir being a part of the company's chain of reservoirs on the Catawba and Santee Rivers. The formation of this impounding reservoir made it necessary to relocate the Catawba Pumping Station on higher ground and at a location several hundred feet south of its original site. The top 10 ft of this reservoir retains approximately 8.25 bil gal of water, thus providing the city with a very excellent impounding reservoir. The drainage area feeding the Catawba at the pumping station site measures approximately 1,800 sq mi. The USGS records, over a period of 13 years, established a daily average discharge of the Catawba River at the pumping station site of approximately 1,560 mgd.

In 1924, a new treatment plant and pumping station was built at Beatties Ford Road and the Seaboard Railway. This plant had a capacity of 8 mgd. Rapid sand filters of modern design were installed in this unit. Electric-driven pumping units were provided for distribution system pressure with gasoline engine–driven standby units provided. A finished-water ground level storage capacity of 3 mil gal and an elevated tank capacity of 1 mil gal were provided with this unit. Distribution system piping was gradually extended throughout the city as required during these years. In 1929, an additional raw-water main, of 30-in. diameter, was installed to supplement the gravity feed of the 24-in. main from the Hoskins Reservoir to the plant.

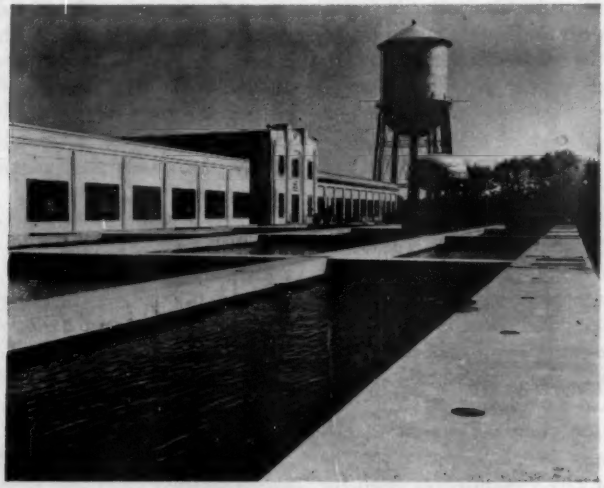

Fig. 1. Vest Pumping Station, Charlotte, N.C.

By 1936, the city was having extreme difficulty in meeting the demand, both in supply and purification facili-

ties as well as distribution system piping, and a program of expansion was then begun to increase facilities in all phases of the water supply, treatment, and distribution system. New electric pumps were installed in existing buildings at the Catawba River Pumping Station, and a 30-in. raw-water supply line was built from this pumping station to the Hoskins Reservoir. This addition provided sufficient raw-water capacity to take care of the proposed additional requirements at the filter plant. Additions to the filter plant and pumping station on Beatties Ford Road were completed in 1939. These additions doubled the capacity of settling basins and rapid sand filters, added 1.75 mil gal to the finished-water ground level storage capacity, bringing the total capacity of the plant to 16.7-mgd treatment capacity and 4.75 mil gal finished-water storage. A new 10-mgd, electric-driven pumping unit was installed to provide additional pumping facilities for distribution system service. At the same time, the distribution system was expanded by the construction of sufficient large distribution system mains to convey the water to the outlying areas of the city. Elevated storage tanks of 0.5-mil gal capacity were constructed in the Plaza and Dilworth areas, and a 1-mil gal capacity elevated water tank was placed in the east central district of the city. The overflow elevations on all these tanks are established at the same elevation. This then provided for 3 mil gal elevated storage, strategically located on four sides of the city. The combined high-service pumping station and filter plant were named Vest Station (Fig. 1) in honor of W. E. Vest, who was for more than 30 years general superintendent of the Charlotte Water Department.

Recent Development

In 1946, immediately after the close of World War II, the rapid growth of the city, combined with the lack of expansion during the war years, created a sudden overload on the then existing facilities throughout the supply, purification, and distribution systems. An expansion program was begun in 1946 to increase all three phases of the water supply system.

Additions were made to the Catawba River Pumping Station in 1948, increasing the size of the existing structures and providing 30 mgd additional pumping capacity. These additions brought the total pumping capacity of Catawba Station to approximately 60 mgd. Surge relief valves and quick-operating cone valves were installed in the discharge headers for the purpose of eliminating danger of a disaster due to flooding of the station. This station was thus provided with all electric pumping units and no auxiliary power standby units. All pumps in this station are designed to operate against approximately 180-ft head. A 36-in. diameter raw-water supply line was installed from the river pumping station to the Hoskins Reservoir, and an additional raw-water storage reservoir of 45-mil gal capacity was constructed at Hoskins, thus providing approximately 100 mil gal raw-water storage in the Hoskins Reservoir, all of which could be distributed to the Vest purification and pumping station.

Additions were made during 1948–49 to the Vest Station, increasing the settling and filtering capacity to approximately 25 mgd. The ground level finished-water storage was increased by 5.75 mil gal, thus providing a total storage at this location of approximately 11.75 mil gal. Additional chem-

ical feeder and chlorination capacity was added to treat properly the full capacity of the plant. Additional electric high-service pumps were added and one gasoline engine–driven auxiliary standby pump was added, bringing the capacity in electric pumps to this station to approximately 60 mgd and the gasoline engine–driven standby unit to a total capacity of 20 mgd. All pumps are designed for operation against an approximate 174-ft head. Peak pumping loads in 1956 reached approximately 42 mgd. At both Vest and Catawba Stations, all the electric units are sized so as to provide a very economical pumping cycle, utilizing various combinations of pumping units.

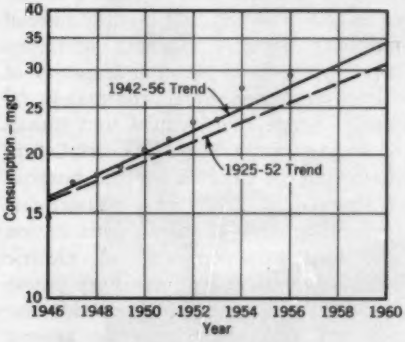

Fig. 2. Maximum 3-Day Pumpages,

Circles indicate average water consumption for the highest 3-day period in each of the years covered.

Fluoridation of the Charlotte water supply was begun during April 1949, using a very closely controlled program of feeding and checking of the quantities of fluoride going into and remaining in the system.

Additional storage space for vehicles and additional stock room and office space were added at the utility shops during 1948–49. All construction, maintenance, repair crews, materials, and supplies are based at this location.

In the 1951–52 period, an additional 36-in. raw-water line between Hoskins Reservoir and Vest Station was installed and placed in service. This line is designed to carry high-pressure filtered water when the Hoskins Plant is placed in service.

In the 1954–55 period, the distribution system was heavily reinforced by the installation of 42-in., 36-in., and 24-in. feeder mains. A 30-in. distribution system main has been installed south of the Dilworth area, and a 42-in. distribution system feeder main has been installed between the Hoskins Filter Plant site and an intersection with a 36-in. distribution system main.

As of 1956, there were approximately 534 mi. of water mains included in the supply and discharge system. Main sizes range up to 42 in. diameter. All mains 24 in. and under are of cast iron or wrought iron. Considerable quantities of prestressed pipe 30-in. in diameter and larger are now incorporated in the system. All pipelines outside the city not now owned by the city will become the property of the water department without further cost to the city whenever the area in which the mains are incorporated is brought within the city limits.

Water is being served now to approximately 180,000 people both inside and outside the city. The average daily water consumption in the fiscal year 1955–56 was 18.23 mgd. The peak pumping rate reached approximately 42 mgd, and the peak-day consumption was approximately 29.6 mgd,

on a date in June 1956 (Fig. 2). Also during the 1955–56 period, the 50,000th water service was installed in the system (Fig. 3).

Financing

All large improvements to the water supply treatment and distribution system have been financed through the sale of serial bonds. Between 1921 and 1956, approximately $15,000,000, exclusive of interest, was or is in process of being invested in improvements for water alone. All these bonds are secured by the general credit of the city and are not classified as water revenue bonds. Interest rates, or yield, on these bonds varied from 5¼ per cent on a 1921 issue to 1¼ per cent on a 1950 issue. The additions at the Catawba River Pumping Station during the 1948–49 period were financed from water revenues and small amounts of revenue have been used occasionally to provide extension of smaller mains. The majority of all capital improvement has been financed by water bonds. The book value of the system has not been accurately determined recently. Estimates are that the book value of the system today is in excess of $20,000,000. The gross revenue for the fiscal year 1956–57 for water alone was established in the budget at approximately $1,750,000. The budgeted anticipation revenue for sewer service charges and surcharges was budgeted during this same period as approximately $434,000, bringing the anticipated gross revenue for operations during the 1956–57 period to approximately $2,184,000. It was anticipated that the operating budget and bond payments for both water and sewers would be within the anticipated revenue during this period.

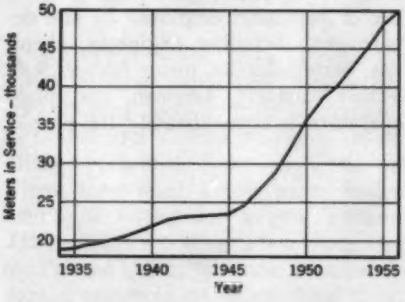

Fig. 3. Water Meters in Service, Charlotte, N.C.

Future Plans

Contracts have been awarded and construction begun on a 12-mgd capacity filter unit and pumping station at the Hoskins Reservoir site. The base building and structures in this unit have been designed for an ultimate capacity of 72 mgd. The present contract includes the provisions of 12-mgd filtering capacity and 6-mil gal finished-water ground level storage. The pumping station will include booster facilities for pumping raw water from the Hoskins Reservoir to the filter plant, as well as a high-service pump for feeding the distribution system. A shortage of funds at this time made it necessary to delete certain parts of the plant. Facilities eliminated and placed on a future contract list were storage facilities of an additional 6-mil gal capacity, a 1.5-mil gal elevated tank, and various pipelines desired at this time to prevent expensive construction and plant shutdowns at a later date.

Plans are underway for the construction of additional large feeder mains into the eastern part of the city and the erection of a 2-mil gal capacity

elevated storage tank. Property has already been acquired for this tank installation, and funds are available for its construction. Plans are also being made for the construction in the near future of a 2-mil gal capacity elevated storage tank in the southeastern section of the city. Feeder mains of the necessary size will be laid to this tank site. In addition, design is underway for the installation of additional 24-in. and 16-in. distribution system mains in other sections, and funds are presently available for this phase of the work.

Every effort is being made to stay ahead of the rapidly increasing demands for water. It will probably be necessary to vote additional bond funds in the very near future to keep ahead of the growing demands of the city and its fringe areas.

Organization

The water department has been operated under the council and city manager form of government since 1930. Prior to that time, the commission form of government was operative. Operating under the city manager, the general superintendent and two assistant superintendents form the top-level organization. There are eight divisions which require annual budgets and special organizations. In each of these divisions, except the administrative division, there is a division head, directly under the superintendent's office. The normal number of personnel employed by the department, including engineers, chemists, bacteriologists, meter readers, collectors, cashiers, foremen, and plant operators, is approximately 160 people.

Development of Master Plans to Meet Future Water Needs

Joint Discussion

A joint discussion presented on Feb. 7, 1957, at the Indiana Section Meeting, Indianapolis, Ind.

Fort Wayne, Ind.—H. A. Kerby

A paper presented by H. A. Kerby, Chief, Water Eng., City Utilities, Fort Wayne, Ind.

MASTER plans have a way of forcing themselves into existence if they have a little encouragement and find a receptive mind. Most utility personnel have thoughts on where they want a few more feeder mains to strengthen a grid system. They visualize adding a few more feet to Tainter gates, or think of another dam, a reservoir, or an overhead tank. Sometimes they get into trouble before obtaining these extra facilities, because they are so busy taking care of matters as they arise, expecting to find a slack period for constructive planning. But the slack period never arrives, and eventually, they have to take time to do the job themselves or engage someone to help.

Most personnel have had enough experience to know what is needed. Those experienced in treating, operating, maintaining, and managing a water utility can set their own goals. It may be necessary to secure consultants to do the designing and the details, but those acquainted with the local needs will have to make the ultimate decisions.

The master plan in Fort Wayne really began several years ago when the ground water source of supply failed to keep pace in quality and quantity with population increase. In 1932, a plant for complete treatment was placed in operation, using the St. Joseph River for raw water. A dam and three feeder mains were installed at that time. Two years ago, the plant capacity was doubled, one more feeder main and two overhead tanks were installed, and more raw-water storage was obtained by building another dam. Future demands, however, will require still more additions within a few years. On a national scale, it has been predicted (*1*) that public water supply demands by 1975 will reach 30 bgd.

Aerial Surveys

Although planning in the future will include still more impounding reservoirs, the principal concern now is to finish a plan on paper for an orderly development of transmission or feeder mains. One of the greatest aids that has been developed in engineering work is the aerial survey (*2*) and the

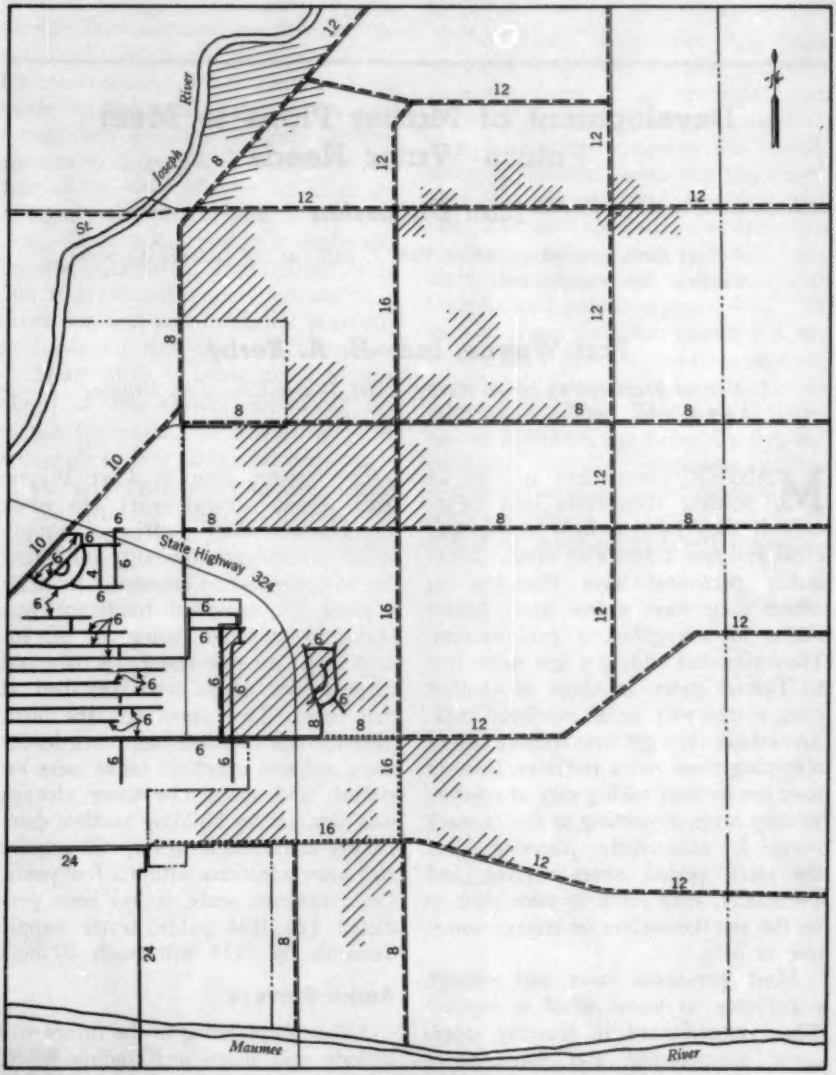

Fig. 1. Master Plan for Development of Area, Fort Wayne, Ind.

Shown in the 11.5-sq mi area outlined above are existing mains (heavy solid lines); mains under contract (dotted lines); and the proposed feeder mains (dashed lines). Shaded portions indicate newly developed sections. Thin lines indicate roads. Figures indicate diameters of mains, in inches.

facilities it has discovered, at what is believed to be a reasonable cost. In 1949, an area of 80 sq mi, including Fort Wayne, was flown and mapped, primarily for the use of the city planner. The mosaic from this work is a ready reference for reexamining city and suburban growth, while the 1,200:1 enlargements give amazing detail for study. A greater usefulness is available through contour maps made photogrammetrically from the contact prints. After the horizontal and the vertical controls are made, a high degree of accuracy is obtainable.

With this new use in mind, another survey was made in the spring of 1956, but included 123.5 sq mi with the proper technique to procure maps at a scale of 1,200:1. The contour intervals are 2 ft and guaranteed one-half contour accurate. Each map shows an 80-acre area, being ¼ mi north and south and ¼ mi east to west. Local personnel provided the vertical control, but had the horizontal control laydown performed by the aerial survey company.

The contour maps are obviously more important in sewer design, but are valuable in routing water mains. They certainly will be used in designing additional impounding reservoirs. Since the engineering for both water and sewer utilities in Fort Wayne is the responsibility of the same department, the opportunity for better planning is apparent. Maps will be made only of areas as they become important for immediate study. The cost per acre is so reasonable, however, that there is a tendency to be slightly extravagant in ordering maps—the cost being about 10–15 per cent of that required for field surveying and office mapping.

Area Development

In order to give substance to the thoughts leading up to this plan, it is in order to review briefly what has gone before. Of the suitable area within the corporate limits of Fort Wayne, 98 per cent was developed by 1950. Most of the new projects lie well beyond the city limits, even though these boundaries have been extended. Installations of water facilities to these outlying sections assumed sizable proportions, both for the utility and the developer. The developer objected to the heavy outlay of cash and the utility wanted the business. Therefore, a plan was devised wherein the utility advances the money for developing the feeder main system and recovers it as the total area is served. It will require a few years of close operation to accumulate enough cash reserve to make this program fully effective, but the program is flexible enough for the utility to vary customer participation and stretch dollars to meet his needs.

Figure 1 shows the first area to come under full master planning for feeder main extensions. This area includes 11.5 sq mi, in an outlying corner of the metropolitan area, and is the most rapidly developing section near the city. All 6-in., 10-in., and 24-in. lines existed prior to this program, while the remainder of the lines are proposed for feeders in the ultimate development of the area. The cost of these feeders will be returned as a small charge per square foot of area served. This charge is now $0.0045 per square foot. In general, 12-in. mains will be located on section lines 1 mile apart and on half-section lines if required. Distribution lines will be installed by the developer or customer

at his expense as needed, but under utility specifications and supervision.

In residential sections, which this area largely consists of, a fire flow of 500 gpm is more of a determining factor than several residential customers' simultaneous demands for domestic use. Consequently, that is used as a basis for sizing lines. There will be adequate capacity to serve several moderate users, as well as meeting the air-conditioning demands.

This design will afford opportunities to connect to area developments lying in other sections. It is important to have more than one feeder to each section in order to maintain service with a minimum of shutdown. These crossties will eventually form a system of more or less concentric loops around the city.

Future annexation groundwork is accomplished at the same time utilities are extended. By requirement, the right of an individual to remonstrate against annexation is waived when he requests water or sewer facilities. This is done by inserting such a clause in the subaddition contract, and all such utility services are covered by contract, or, in the case of individuals, by an appropriate form, notarized and recorded. Through close cooperation with city personnel, the city planner, realtors, developers, and others, there has been gained a comprehensive picture of what can be expected in future growth and requirements.

Planned Street Use

Planned street use, partially the concern of a water department, does involve many facilities. In Fort Wayne, these are water mains, sewers, city power, a private power company, the telephone company, the gas company, signal department, traffic engineering, transit company, street department, fire and police divisions, and sometimes state highway department, as well as the public. It is hoped to do a better job of using the streets and thoroughfares by planning ahead. To this end, the administration has created a city of Fort Wayne engineering center, where these appropriate uses will be coordinated and administered. Cooperation between various engineering departments will be facilitated, so that a better job of meeting not only water requirements but other needs as well will be done.

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Michigan City, Ind.—*H. J. Draves*

A paper presented by H. J. Draves, Supt., Dept. of Water Works, Michigan City, Ind.

Michigan City, Ind., is located on the south shore of Lake Michigan in the heart of the dunes. Due to its geographical location and its fine beaches, it caters to many vacationers throughout the summer months. Michigan City and its sand beaches have often been referred to as the "Atlantic City of the West." Although the city is blessed with many assets, its water utility is again suffering from growing pains.

With Lake Michigan located at its front door, Michigan City obtains its

water supply from this abundant source. There are two cast-iron intakes—one a 24-in. and the other a 42-in. main—through which water flows by gravity to the suction well. Both intakes extend into the lake approximately 3,000 ft, at which point the water has a depth of 35 ft. Temperature of the raw water varies considerably, depending mostly on wind and current direction. During the winter months water temperatures will be slightly above 32°F, whereas during the summer months it may reach a temperature of 80°F.

The population of Michigan City according to the 1950 census was 28,395. Today, it is 31,663, an approximate 11 per cent growth, while the water demand increase is approximately 50 per cent for the same period. Some of the increased consumption can be attributed to the modern appliances; a large factor is the water being used for air conditioning, which has had a considerable growth during the past 5 years.

The connected customer total increased from 6,620 to 8,202 during the 5-year period from 1950 to 1955. This, in itself, presents a problem, as there are not any large feeder mains to supply the grid of 4-in. and 6-in. pipe in the outlying areas. The real problem with Michigan City is that it is bottled in on three sides. Lake Michigan is to the north, Beverly Shores is to the west, and Lakeland to the east, so the only direction to which Michigan City can expand is to the south. This presents a considerable pumping load, considering that the plant is located on the extreme north end of the city. To add to the difficulties, a large leak, through which a large amount of water was being lost, was recently discovered in the 30-in. supply line, which crosses the harbor.

Although the last expansion program was completed in 1952, plans are under consideration for a new, long-range expansion program to meet anticipated demands.

Need for Improvements

Michigan City is no different than any other community having a municipally owned water utility. Rates throughout Indiana must be approved by the public service commission, and this also applies to privately owned utilities. A privately owned utility, however, is entitled to a fair return on its investment, and rates are based accordingly, while a municipally owned utility's investment is not considered when establishing its rates.

Experience has shown that the public service commission only provides rates to cover the cost of operating and maintenance, depreciation, and amortization. Nothing is provided in the rate schedule for main extensions or plant expansion; consequently, a reserve cannot be accumulated for plant expansion purposes, which makes it difficult to operate as a self-sustaining utility.

Most of the municipal utilities are controlled by local governments, and such controls may make it difficult at times to obtain adequate funds for operation and maintenance, let alone funds for improvements involving enlargements. So the only recourse, invariably, is a bond issue to finance improvements.

The Michigan City utility has been operating on a hand-to-mouth basis for a number of years. The last expansion program consisted of plant and distribution system, in 1951–52, and in order to hold the bond issue below $1,000,000, it was possible only to do a partial job. The survey for the expansion and improvement program was made in 1950, and the indicated daily

average rate of increased demand on the system was approximately 0.8 mgd during the period of 1940–50. Maximum demands were approximately 14 mgd. Therefore, the filter capacity was increased from 8 to 12 mgd, low-head pumping equipment was added, the distribution system improved, and a new feeder main installed.

The accelerated rate of increase during the past 5 years in the demand placed upon the water plant system has again taxed it to capacity. Pumping records show an approximate 50 per cent demand increase since 1950. Average pumpage for 1950 was 4.59 mgd, while in 1955 it was 7.36 mgd, an increase of more than 62 per cent. Peak demands of 18 mgd have been reached and that figure would have been exceeded if there had been additional plant capacity.

There are two elevated tanks on the distribution system, a 1-mil gal tank, and a 0.75-mil gal tank. During the summer months, the 1-mil gal tank is emptied practically every day. The smaller tank, due to a slight variation in elevation and its location in an area where the load is not quite as heavy, is lowered to the extent where elevated storage is negligible. Pressure tests, run during peak load periods when the tanks are practically empty, have shown that the distribution system is so seriously overloaded under maximum demands that system pressures as low as 12 psi have been recorded. With all of the high-head pumps in operation, a serious conflagration at that time certainly could be a fatal blow to not only the water department, but to the community as well, especially from an industrial growth standpoint.

The above figures show conclusively that the present plant and system has reached its maximum capacity at 12 mgd, and although for short periods this rate is exceeded, there is no reserve capacity to meet the ever-increasing demands made upon the plant and system. The system is, therefore, incapable of providing adequate fire flow and pressure during the high-demand period. In addition, as the growth of the city expands more and more at the perimeter areas, fire flow and pressure at many points are poor, even during average- and low-demand periods. At times, all of the high-head pumps are operating around the clock, leaving no standby equipment; this cannot be considered good business practice, even in a municipally owned plant.

Preliminary Plans

The present plant and system at Michigan City can no longer cope with the demand. Since 1953, the overloaded plant and system capacities have been noted in annual reports to the board of trustees, but like many governing bodies, it treated the situation too lightly.

During the past several years, the city has received a number of inquiries regarding the possibility of serving new industries. Industries do not locate in the center of a community, but in the outlying areas, where there is sufficient acreage to construct their plant and allow for future expansion. Practically every inquiry received a reply that the city did not have the capacity to serve them, and that there were no supply mains in the areas selected. This resulted in the loss of those industries to neighboring cities, which soon made many people come to the realization that something had to be done to meet these demands.

In a 1955 report, the author again stressed the importance of a plant expansion program, and asked the board to give consideration to the problem. The board was in complete sympathy with the plea, for it, too, felt that the community could ill afford to lose these industries due to the lack of an adequate water supply. At this time there was criticism heard over the lack of an adequate supply to serve industry.

The city then called in an engineering firm to discuss the advisability of a complete survey of its system, including requirements for intake, low- and high-head pumping, filter plant expansion, and feeder mains to provide a loop feeder system to serve from the periphery of the city. The necessary action was taken by the board to enter into a contract with the engineers to make a complete survey and recommendations of what would be required to meet the increasing demands of the past few years. A contract was also entered into with the water department's attorney to handle legal details.

In due time, the engineering firm submitted a report of its findings. The report is very complete in detail, covering all aspects of the expansion program. This report was used as a basis for convincing council officials of the city's needs.

Final Action

Although the utility operates under a state statute, the board of trustees is appointed by the mayor and must obtain authorization by resolution of the common council and its concurrence on the water department proposal to proceed in planning needed extensions and improvements. These are ultimately to be completed and paid for through the issuance and sale of revenue bonds of the department, under authority of an ordinance to be adopted subsequently by the council, when all necessary engineering, legal, and financial data are completed.

The community has never before issued bonds in an amount in excess of $1,000,000, and facing the council with a $2,765,000 project requires a well-organized plan. The utility did not do this hurriedly, but planned every move until it was sure that it had a convincing answer to any question that might arise.

At an informal meeting with the mayor and common council members, the utility proceeded carefully, step by step, to explain everything in detail. At this meeting, the board of trustees also asked the common council for its concurrence with the board in making application to the Community Facilities Administration (a US agency created under Public Law 560) for an advance of funds, interest free, to enable the board to proceed in obtaining detailed plans, specifications, and data concerning the required extensions and improvements. Any such advance obtained from the agency would not be a loan or indebtedness of the water department, but would be payable from proceeds from the sale of the anticipated revenue bonds at the time when construction on the proposed project is commenced.

The council members soon realized the importance of an expansion program, and were very receptive. As soon as they became acquainted with the problem, they authorized the attorney to prepare the necessary resolution for their consideration.

The resolution was passed on Feb. 18, 1957, and no time was lost in preparing and making application to the

Community Facilities Administration for an advance to finance the detailed plans and specifications. This application was made to the regional office, processed and approved by that office, and forwarded to Washington for final approval.

In order to get some action in expediting the application, an emergency clause was inserted in the appeal for action. The aforementioned leak in the 30-in. transmission main under the harbor between the pumping station and the city was the basis for the clause. It caused an increase in the unaccounted-for water for 2 years, the total becoming of such importance (16 per cent for 1957) that engineers were engaged to traverse two of the harbor-crossing mains, a 20-in. and a 30-in. line, by establishing pitot stations on both sides of the harbor. It was determined that the loss was approximately 1½ mgd on a 5-mgd flow, through the 30-in. line. The 20-in. line tested out satisfactory.

Emergency repairs on the 30-in. line are quite costly, due to the fact that this line is 25½ ft below the water surface, of which about 20 ft is silt and mud. By impressing the officer in charge at the regional office of the Community Facilities Administration of the importance of this emergency the utility was advised on Sep. 17, 1957, that the application had been approved by the Washington office.

Because of the official approval of the application for an advance of funds, the board of trustees has authorized the engineers to proceed with detailed plans and specifications. It is estimated that it will take approximately 2 years for the completion of the program.

On a national scale, approximately $650,000,000 was expended for water utility construction in 1957. This clearly indicates that Michigan City is not the only place where the community has outgrown its water utility.

Water Supply Developments in South Dakota

Charles E. Carl

A paper presented on Sep. 26, 1957, at the North Central Section Meeting, Fargo, N.D., by Charles E. Carl, Director, Div. of San. Eng., State Dept. of Health, Pierre, S.D.

ON Oct. 20, 1933, W. W. Towne, who was director of the Division of Sanitary Engineering, South Dakota State Board of Health, appeared before the existing Minnesota Section of the AWWA to discuss a typhoid epidemic that had occurred at Chamberlain, S.D., in December 1932 and January 1933 (*1*). In this epidemic there were 282 cases and 29 deaths, and the epidemiological investigation results indicated that the municipal water supply was the one possible source of infection common to all patients. Twenty-five years have elapsed since this epidemic. Many people have forgotten it, and many feel that such outbreaks are a thing of the past, but it should not be forgotten that 2–5 per cent of persons having typhoid fever remain carriers, some for the rest of their lives.

There are still in South Dakota, and probably in other states also, public water utilities that are not properly protected to assure the safety of the water when exposed to a source of virulent infection. For instance, as recently as the week of Sep. 9, 1957, one district engineer reported a situation based on an investigation of a city which had interrupted its schedule of submitting bacteriological water samples. It was found that the community's well had failed—a well drilled without the advice of a consultant and contrary to the recommendations of both the state geological and health agencies—and the community was pumping untreated lake water into the city distribution system for fire protection. It is a safe bet, at considerable odds, that this same lake water is being used for domestic purposes also. These situations and the poor health judgment of the people responsible for such actions become even more appalling when the record of the 37 cases of reported typhoid fever (no deaths) in South Dakota from Jan. 1, 1951, to date is taken into consideration.

In spite of occasional reverses, however, such as the small community pumping untreated lake water, public water supply sanitation in South Dakota is progressing. South Dakota, with a population of about 700,000, has about 240 public water supplies including nine community wells with no distribution system, serving 365,000 people. Since Jul. 1, 1954, the following improvements have been made: seven new water supply systems; three new systems authorized or under construction; 66 new water supply sources, in 52 communities; seventeen major remodeling projects; eight improved storage facilities; and four fluoridation installations, for a total of seven.

Quality and Supply

There has been no apparent tendency for municipalities to convert from

well supplies to surface supplies, but there has been a notable tendency to try for a better quality of water. Many geologic formations in South Dakota furnish hot waters (up to 130°F); high solids (up to 3,700 ppm); hardness (up to 1,800 ppm); high sulfates (up to 1,400 ppm); high chlorides (up to 1,500 ppm); high fluorides (up to nearly 7 ppm); and iron and manganese (up to nearly 8 ppm). Some South Dakota well waters are wet but beyond that can hardly be called water as far as domestic consumption is concerned. In July 1955, a mobile saline water conversion test unit was moved to the town of Miller. The activity was a part of the saline water conversion program established by congressional legislation and was under the administration of the Department of Interior. The test unit used the electrodialysis method for mineral removal, processed approximately 1,500 gph and reduced the total solids from about 2,000 ppm to about 350 ppm. A report of the study has been published (*2*).

The work at Miller was experimental, but several cities are moving realistically to obtain better quality water in their distribution systems. Sioux Falls constructed a $1,100,000 chemical treatment plant for iron and manganese removal in 1955; Brookings has just let contracts totaling $174,400 for a similar plant; and Madison is voting on a bond issue, $395,000, for comparable treatment. Many of the smaller cities have pressure filter iron removal units, and some are trying out contact filter manganese removal units, but as yet the South Dakota Department of Health is not convinced that these latter will work satisfactorily. Other communities are trying polyphosphate treatment for high iron and manganese waters, but this has been generally unsatisfactory if the iron and manganese content exceeds 0.5–1.0 ppm.

Many South Dakota cities are short of adequate sources of supply, storage, or transmission line capacity to handle peak loads. A notable exception is Pierre, the capital city. Pierre now has more storage than the peak 24-hr period of consumption, which to date has been 4.84 mil gal for approximately 9,000 people. The system can produce 6.05 mgd, and there will be 6.0 mil gal in storage. In addition, during these periods of general water shortages, water restrictions, and tendency toward higher rates, Pierre annually gives each of its consumers 7,000 cu ft of free water during the months of June–September.

Missouri River Development

More than 90 per cent of South Dakota is in the Missouri River Basin. Consequently, the Missouri River Basin Development Program is of paramount interest to those working with water resources every day. The Missouri River just about divides the state east and west until it reaches the Nebraska border, where it then flows east toward Sioux City, Iowa. Streams west of the Missouri River generally flow east to their confluence with the river, and those drainage areas east of the Missouri River generally flow south to their confluence with the main stem. There are now completed, or under construction, three major impoundments for the Missouri River in South Dakota, and a fourth is under investigation. These start about 5 mi upstream from Yankton in the southern part of the state, and the farthest upstream dam, Oahe, 6 mi north of Pierre, will create a pool approxi-

mately 200 mi long, extending nearly to Bismarck, N.D. The Missouri River will be one series of reservoirs across South Dakota. It is not the purpose of this article to discuss high dams and low dams, public power and private power, or power distribution and land use for reservoir development. Many problems, however, are imposed on municipalities and public health offices, but on the credit side, a tremendous potential is available from the mass of water stored in the reservoirs.

Municipal water utility and waste disposal practices on the Missouri River have been, or are being, changed. Two communities' water plants will be flooded, one of which (Chamberlain) has had a new plant in operation over 1 year and the other (Mobridge) has a new plant under construction. The Cheyenne Indian Agency headquarters will be flooded. A town of approximately 400 people, Eagle Butte, to which the agency headquarters will move, is just completing a new water system and a new sewerage system, including treatment.

Problems have arisen in supplying water to the municipalities on the river, because of fluctuating flows with the construction program. In Yankton, during the closure of Gavins Point Dam, a high-capacity, low-head pump was floated on the river outside of the plant with flexible lines to the preliminary settling basin. In Mobridge, extra water was released from the upstream dam, Garrison, increasing the river depth sufficiently to protect the raw-water intake pipes. While writing this article, the author was contacted by the Army Corps of Engineers with respect to contemplated water supply problems when the Missouri River flow is shut down for closure of the Oahe Dam sometime in 1958.

Studies have been proposed relative to supplying Missouri Basin reservoir water to municipalities located long distances from such a source of supply, but to date funds have not been budgeted for this purpose.

South Dakota is cooperating with the Corps of Engineers and the USPHS central Missouri River water quality investigation program, and has a network station at Yankton on the Missouri River cooperating with the national raw-water quality basic data program.

In addition to construction on the Missouri River, work is proceeding on tributary streams. Shadehill Dam on the Grand River, about 12 mi south of Lemmon on the South Dakota–North Dakota border, has facilities for Lemmon eventually to obtain water, since presently this community has a well field. Pactola Dam has just been completed on Rapid Creek in western South Dakota, about 20 mi upstream from Rapid City, a popular tourist center and the location of Ellsworth Air Force Base. Rapid City has purchased storage in the Pactola Reservoir, even though its present supply is from wells and infiltration galleries. The city is currently studying future water developments.

Water Law

The problem of water resources, water rights, equitable distribution of surface and ground waters, water use, and water conservation plagued the state for many years, and all of the answers still have not been found. Many are of the opinion, however, that South Dakota is moving in the right direction with its new water laws. After several years of investigation

and study, including a revised water law in 1953, which the governor would not sign, the South Dakota legislature in 1955 revised the state's water laws in their entirety. In part, the revised laws provided:

1. A commission representing various interests was to be appointed by the governor, and the commission could hire an engineer and other personnel as needed.

2. In the matter of priorities, domestic use has the highest priority, and the next highest is municipal use.

3. In the matter of water rights, vested rights can be established, and a municipality can be authorized an appropriation of water for future use (the present rule-of-thumb estimate of the commission is to allow for future appropriation of approximately three times the present use). Actual future appropriation is based on estimated per capita consumption and population.

4. Well drillers have to be licensed and keep complete records on wells drilled. A permit is needed prior to drilling a new well. The commission will issue construction standards, and review the spacing of wells.

The commission has hired an initial staff of three engineers, and the program is well underway. Between the commission and state health officials, there has been an excellent working relationship through respective interests in water rights for municipalities and municipal well construction. The problem yet to be resolved is the insistence by the department of health that plans and specifications for municipal wells be prepared by engineers practicing in accordance with the South Dakota registration law, whereas the water resources commission issues permits for wells on application by drillers.

Public Law 566, for watershed protection and flood prevention, administered by the US Department of Agriculture, will probably be of material assistance to certain South Dakota communities experiencing water shortages. One project, in the development stage, includes the town of Beresford, which has requested consideration with respect to including water supply storage in the pool development.

Conclusion

It may be said that the main problems of water supply in South Dakota and future developments fall into four categories: [1] procurement of adequate supplies for community development; [2] the continued necessary improvements in treatment and quality; [3] problems arising from the development of the Missouri River Basin; and [4] program expansion and development under the new state laws pertaining to water use and rights.

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Factors in Evaluating Water Utility Securities

Grant A. Martell

A paper presented on Oct. 10, 1957, before the Pennsylvania Water Works Assn., Atlantic City, N.J., by Grant. A. Martell, Second Vice-Pres., Investment Dept., New York Life Insurance Co., New York City.

AN institutional investor considers many factors in evaluating the investment position of securities of water utilities. Each case presents special problems, but only those factors which are generally given the most weight will be considered.

Source of Supply

Adequacy of source of supply is a major factor on which all available information is sought. If the utility has a record of never having curtailed water service for lack of supply, if there is little flat-rate business, and if the system does not seem to be faced with the prospect of rapid growth in demand, the institutional investor may be satisfied fairly easily on this point. On the other hand, if there has been a record of curtailment of water service during dry periods in recent years or if the service area is growing rapidly and has prospects of marked increased growth, strong reassurance is required as to water supply. The investor always wishes to know the plans of the management for augmenting its raw water supply during the next 10–15 years and what steps have been taken to insure that this can be done.

The problem of water supply seems to be approaching almost national proportions. The indiscriminate drilling of water wells, with close spacing in suburban residential areas by real estate promoters and home builders is, obviously, not the answer to the problem. The development of many new surface supplies, the artificial recharging of ground water supplies, the reduction of evaporation losses from lakes and reservoirs, the reuse of water by large industrial consumers, and, eventually, the conversion of sea water to fresh water will become increasingly necessary over the coming decades.

Treatment Facilities

If there is a good well supply and if treatment is limited to chlorine or other chemical disinfection, then adequacy of water treatment facilities is not a matter for concern. On the other hand, if the water utility uses a surface supply which is subject to serious bacterial or industrial pollution, the investor must be assured as to the adequacy of the filtration and treatment plant. It goes almost without saying that the plant effluent must be potable; without undue turbidity, color, or hardness; and free of unpleasant tastes or odors.

It is reassuring to see a well equipped laboratory manned by a graduate chemist and bacteriologist. In smaller water utilities, this may not be feasible, so there must be assurance that adequate sampling is done and that analyses are being made by competent outside technicians. An

investor may want to examine the records of these tests of raw water and plant effluent, as well as pertinent reports for recent years of the state board of health.

Distribution

In the area of transmission and distribution the factors considered are size and type of mains and whether or not they form a good gridiron. A high percentage of cast-iron pipe is preferred, and present feelings about asbestos-cement pipe are unsettled. It is understood that the use of asbestos-cement pipe for water purposes is growing faster than cast-iron, steel, or reinforced concrete pipe but it is recognized that one type of pipe is not ideal for all conditions. The use of cement-mortar lining for cast-iron pipe and fittings, cement-mortar protective coatings for large diameter steel water pipe, and the use of flexible joints for cast-iron pipe are all developments of interest and significance to the investor.

That adequate pressures are maintained—particularly at high elevations—and that there is ample, dependable pumping capacity to meet peak demands are points on which assurance is sought. A schematic map showing the elevations of each of the important structures of the system and a record of average pressures at the pumping station and at the system extremity are very helpful in satisfying interest on this point. Assurance is needed of the adequacy of raw-water and finished-water storage capacity, and the record of unaccounted-for water is also important.

Maintenance

Adequacy of maintenance is tested from the standpoint of the income account—that is, the annual dollar expenditures for maintenance for several years in relation to operating revenues; more helpful is an actual inspection of the above-ground properties. Although obviously not conclusive, the state of housekeeping is often assumed to be indicative of the condition of the properties.

Economic Conditions

The economy of the territory served is considered very important. In order to form an opinion of this, a field trip is usually required, unless the investor is already familiar with the area. If a recent economic survey of the territory served is available, it should be supplied. A tabulation of annual dollar and gallonage sales for several past years for each large industrial customer is also helpful.

Because the institutional lender is a long-term investor, it is generally not interested in a water utility which serves an area dependent on a single industry—particularly when that industry is based on a depleting mineral. A well balanced economy is rated as an important favorable factor.

Population trends in the territory served are also studied and a static or declining population trend is considered a major unfavorable factor.

Management

The ability of the management and depth of management are important considerations. Obviously, with a small water utility one cannot look for great depth of management, but it is reassuring to see a younger man advancing who will be competent to take over the running of the company in time.

Earnings Record

The earnings record—for a period of 10 years or more—is carefully studied and any important deviations from

normal of any of the income or expense items requires an explanation. It is expected that pro forma fixed charges will have been covered at least twice after depreciation but before income taxes.

The balance sheet should show that the pro forma funded debt does not represent an undue percentage of total capitalization or of net plant account. The ratio of mortgage debt to capitalization must not be more than 75 per cent, as a higher ratio would present a valuation problem for investors who are New York life insurance companies. If the plant account is carried on a basis which is justified by the valuation ideas of the public service commission having jurisdiction, the preference is that the ratio of total funded debt (including junior funded debt) be less than 70 per cent of total capitalization.

When a preferred stock is being analyzed, there is taken into account, among other things, the ratio of total funded debt plus preferred stocks to total capitalization. This ratio should not be higher than 80 per cent, indicating a common stock and surplus ratio of at least 20 per cent.

In order to meet the legal requirements for life insurance companies in New York state, common stocks must, among other things, be registered on a national securities exchange, which eliminates for purchase almost all water company common stocks by such investors.

Regulatory Climate

As with other public utilities, considerable importance is attached to the regulatory climate—particularly when an equity security is being analyzed. It was disappointing, for example, to see the Dec. 12, 1955, decision of the Pennsylvania Public Utility Commission in the case of the Citizen's Water Company of Washington, Pa. This was concerned with the handling of accelerated depreciation for rate making purposes and, it will be recalled, the commission did not permit normalizing of earnings for rate-making purposes—which has the effect of passing on to consumers all such income tax savings. This decision of the commission was reaffirmed in the Manufacturers Light & Heat case.

It has been noted with favor that in the recent People's Natural Gas case, the Pennsylvania commission did not force that company to minimize income taxes for ratemaking purposes when the management elected not to take advantage of the accelerated depreciation options.

The low rates of return generally allowed to water utilities by regulatory authorities is a matter for concern as it encourages high debt ratios and makes difficult the sale of equity securities. This weakness is compounded by the fact that many water utilities do not realize anywhere near their theoretically allowed rate of return. According to an investor's service,* the indicated realized rate of return (the ratio of operating income to net plant) of operating water companies has been in a declining trend for more than a decade and was only 4.60 per cent in 1956.

In the Aug. 5, 1957, decision of the California Public Utilities Commission in the California Water & Telephone Company case involving its Monterey water division, a rate increase was allowed which was calculated to produce a 6.5-per cent rate of return, whereas a return of only 5¾ per cent was allowed this same division in 1952. This

* Moody's Investors Service, New York City.

favorable rate of return determination was partially offset by the dim, though tentative, view the California commission took on normalizing earnings for accelerated depreciation.

The utility must own, without burdensome restrictions, franchises, water rights, licenses, and permits sufficient to carry on its business.

Stock Ownership

Unless the common stock of a water utility is held by a strong holding company, it is considered an adverse factor if there is not enough public ownership of the stock to make possible a fair market for the issue. The reasoning is that as the company grows there will be need for raising new common-stock equity, and if the common stock is closely held by interests unable to provide additional funds through purchase of common stock, the company's financing and growth may be seriously hampered.

Mortgage Provisions

As is known, the principal financing vehicle for water utilities is the mortgage—which should be a good deal like a man's legs. It will be recalled that in the famous Lincoln-Douglas debate, Lincoln made reference to the length of a man's legs. On questioning by Senator Douglas, he said that "a man's legs should be long enough to reach from his body to the ground." In other words, the mortgage should prevent the investment position of the bonds from being greatly impaired by the action of an improvident management—should it come into power. On the other hand, the mortgage should not be unduly restrictive. The following are a few of the major provisions which should be included in a water utility mortgage:

1. Limitation of issuance of additional bonds to not more than 66⅔ per cent (preferably 60 per cent) of net property additions—measured from a reasonable peg date—provided pro forma interest charges have been earned at least 1¾ times after depreciation but before income taxes (By "net property additions" is meant gross property additions minus retirements since the peg date.)
2. A covenant or, better still, a maintenance fund which would insure that 10–12 per cent of revenues will be expended for maintenance or reserved for depreciation
3. A small sinking or improvement fund of 1–2 per cent annually of the maximum amount of mortgage bonds outstanding
4. A mandatory call of bonds in the event of acquisition of all or of a major portion of the properties by a governmental body (In this circumstance it is fair to provide for a lower call price than in the case of refunding. Under the present conditions of the capital market, the institutional lender is unwilling to purchase bonds unless he is assured that they will not be called for a period of 5–10 years or more. Some call formulas provide that this limitation would apply only in the case of refunding at a lower interest rate.)

Cost of Capital

Cost of capital is, obviously, of major importance to managers of water utilities because of the heavy capital investment in and the slow turnover of plant. As is known, it requires about 9 years for revenues to equal the gross plant account of the average water utility. This is a serious handicap when it is recognized that the electric industry turns over its plant account in about 4 years, the

telephone industry in about 3 years and the gas industry in about 2.6 years. This means that the operating ratio of a water utility must be very much lower than that of other utilities if an adequate return on investment is to be realized.

The following are suggested methods for reducing the overall cost of capital. Where possible, small companies should be combined with neighboring water utilities to make larger operating units. This will make the bonds and preferred stocks more attractive to the institutional investor. A larger company would result in a larger issue of common stock, which should result in a broader market and a higher price for that stock. This, in turn, should result in a lower cost for new capital as required. This might very well result in substantial operating economies. It would certainly reduce, relatively, the legal and related expenses in connection with new financing. At the end of 1955, Pennsylvania had 406 privately owned water utilities—more than any other state. Of these 406 utilities, 333, or 82 per cent, had annual revenues of less than $50,000; 61 had revenues between $50,000 and $250,000; and only 12 had revenues of over $250,000. This would seem to indicate that there are many opportunities to combine very small operating units into larger ones.

A utility and its management should be made known to one or more institutional investors who are interested in purchasing water company securities. Do not wait until the money is needed to do this.

The efficiency of the various divisions of operation should be improved. This will be reflected—sooner or later—in higher earnings. Obsolete equipment should be eliminated and housekeeping should be improved in pipe yard, stock room, plant, and office. Frequent painting can be effective, too.

The realized rate of return should be observed carefully and there should be no fear in asking for a rate increase if one is justified. Security holders should be kept advised of current developments—good as well as bad—for security holders like to feel that they know what is going on.

Improved customer relations may also pay good dividends when a rate increase is sought during franchise renewal, or when some concession is needed from the local authorities. To most customers, the water company employee with whom they come in contact *is* the water company and he or she can be a powerful force in influencing public opinion.

The accounts of a company should be audited annually and its annual financial statements certified by a public accountant. These audited annual financial statements should be furnished to interested lending institutions and insurance companies may be supplied with the information called for in the standard "Confidential Report for Insurance Companies."

Outlook

As is known, long-term interest rates are the highest they have been for more than 20 years. This has resulted, basically, from the fact that the aggregate current demands for long term funds is larger than the capital market can provide under present conditions. Interest rates will continue high as long as this large inequality continues to exist.

Federal Reserve Board policy at this time favors high interest rates as a means of curbing the price inflation

that is taking place. Obviously, should deflation set in, the Federal Reserve Board would quickly reverse its policy and the effect on interest rates might be quick and important.

It is the author's view that for the next 12 months, long-term interest rates will continue high although the peak for some time to come may well have been reached.

Conclusion

Since the supply of funds in the United States is currently insufficient to meet the recently developed explosive demand for long-term capital funds, there is probably no lending institution that is welcoming new borrowers with open arms at this time. The supply-demand relationship for capital funds, however, as with other commodities, is subject to change.

Increasing population and the growing per capita use of water in the United States indicate heavy capital outlays by water systems—public and private—over the coming years. In view of the present and prospective future heavy demands for capital funds by other issuers of securities, the water industry must make itself as attractive as possible to both institutional and individual investors and be prepared to offer yields and protective provisions which will be competitive with those offered by alternative investments.

An unavoidable amount of legal and related expenses accompanies the sale of senior securities to an institutional investor; certain legal opinions must be given, a title search must be made, and printing expense must be incurred. Because these expenses do not vary greatly with the size of the issue, it may cost an issuer less to sell securities locally when the amount is small—perhaps less than $100,000.

The water industry has many favorable factors: it represents the largest transportation industry in the country; it supplies a product which is indispensable; competition is virtually nonexistent; it influences the location of factories, businesses, and homes; most of its assets have a very long service life with obsolescence playing a very minor role; and its earnings are relatively stable.

The principal problems are to provide for future expanding water needs, to earn a fair rate of return, to adapt financing plans to the radically changed state of the capital market, and to let the public know about the industry's good works.

Financing Small Water Utilities in California

Allen D. Harper

A paper presented on Nov. 1, 1957, at the California Section Meeting, San Jose, Calif., by Allen D. Harper, Vice-Pres., Securities Dept., Pacific Mutual Life Insurance Co., Los Angeles, Calif.

CALIFORNIA is the home of the most rapidly growing, privately owned water utilities in the country. There are, in this state, probably a larger number of strong, independent, relatively well financed water utilities than in any other comparable area in the country. At least eight privately owned utilities serve more than 25,000 customers—which is equivalent to furnishing service to a city of 75,000 population. These utilities generally have property investments of at least $7,000,000 with gross revenues of $1,200,000. This is no longer only small business. The following observations however are directed not only to this relatively financially sophisticated group but also to that much larger body made up of small utilities. Their financial problems essentially have developed as a result of exceptionally fast growth, inflation, and increasingly "tight" money. These three major factors have, in fact, been affecting all business and especially all public utilities.

An example of extreme expansion is one of the water utilities in Southern California which has expanded services from roughly 2,000 in 1947 to 36,000 at present and investment from $500,000 to $12,500,000. Its management has been faced with serious problems concerned with new capital, new and adequate sources of water supply, personnel, development of accounting systems, public relations, and maintenance and development of improved engineering standards.

Growth of utilities in outlying areas has been particularly great, for almost all California cities have spilled out into the suburbs and rural areas. Most of these areas are served by privately owned utilities, by mutuals, and, in some instances, by county water districts. Since the growth is not ended, this process will continue to affect many. It is almost impossible to list adequately all individual problems. It can be pointed out, however, that the effect of growth is to emphasize the adequacy of long-range planning. Planning means adoption of higher standards of engineering, the development of adequate sources of supply, and, probably, greater interconnection not only between the various parts of one water system but also between various adjacent properties.

Higher costs affect all business and, particularly, public utilities which necessarily must spend large amounts of money to meet expanded requirements or merely to continue replacement of worn out installations. Average capital outlay per customer continues to inch slowly upward. It is recognized clearly that many utilities have been unable to foresee accurately the extent of the expansion which has oc-

curred in their area. Conservatism in estimating growth has probably led many to design inadequate systems. If this results in a subsequent replacement of small lines to provide additional capacity, clearly there is waste. Some is unavoidable, of course, but careful engineering procedures should be developed.

Sources of Capital

Since 1946, there has been, generally, a steady movement toward a higher cost of money. Although money is still relatively cheap in this country—even when compared to earlier levels of interest rates such as existed here in the 1920's—to be concerned with past, extremely cheap money will accomplish nothing for those who need money under today's conditions at current interest rates. The problem which many have is actually less one of interest rates and more one of finding an adequate source of long-term capital. It is basically where to seek capital and what methods should be used to attract it. It should be understood that some of the following suggested solutions will not be appropriate in all circumstances.

Sources of capital available to any privately owned water utility are common equity, preferred stock, donations, construction advances, bank loans, and long-term debt.

Common equity in the smallest company usually represents the capital of an individual owner and that contributed by his family or friends. Retained earnings are relatively unimportant as a source of capital because company growth is usually too fast to permit accumulation of significant sums by this route. New common stock is required and, generally, must be sold to local customers, friends, and individuals conversant with the company and its service area.

Preferred stock is also frequently sold by small companies to their customers. This source of capital is generally quite limited but very important to individual companies. Clearly, sales of preferred stock to customers depend to a large extent on the local reputation and standing of the water utility's management. Only a relatively few of the larger utilities in California have been able to establish a truly marketable preferred stock in the industry.

Donations arise largely from expansion operations which are in themselves entirely or partially uneconomical and for which customers are required to provide all or part of the capital needed. This cannot normally be regarded as a source of long-term capital.

Construction advances are somewhat comparable to donations except that construction advances generally are required from subdividers in connection with development of properties which will be very desirable from an economic standpoint when all or substantally all of the anticipated customers are connected to the utility's lines. Some utilities have used construction advances very extensively as a means of financing their rapid expansion. In some instances this has been seriously overdone. Certainly, under prior methods of repayment of the construction advances, those making extensive use of the 10-year, 35 per cent of gross revenue repayment method were dedicating a large percentage of future gross revenue to debt service. Fortunately, this method is now probably completely abandoned.

The present method of repayment of such construction advances from 22 per cent of gross over a 20-year period

is also probably too severe a requirement on the water utility. Under present commission rules the monies received under construction advances are not considered as part of the rate base. Therefore, a water utility is theoretically not given any earnings from these properties except the amount equal to depreciation on them. It would appear that the 22 per cent of gross revenue repayment may require the company to pay back all its cash earnings derived from the particular properties constructed out of funds received under construction advances. A water utility, then, which has built a substantial portion of its properties with funds of this type would find itself with a severe cash shortage because earnings could easily fall short of supporting normal operations, debt service, actual replacement requirements, and return of capital to the subdividers. This device of requiring subdivider to provide, in effect, substantially all the money for expansion should be reviewed most carefully in relation to total capital obligations of the particular industry.

There have been a number of instances where companies exchanged the 3 per cent preferred stock at par for these construction advances. This, apparently, has been very desirable from the standpoint of the privately owned water company for, after the construction advances have been traded for preferred stock, the properties become part of the rate basis. Thereafter, the preferred stock dividends on the new stock actually are less of a cash drain than was the refund of 22 per cent of the gross revenue. There will be little gain to the water utility unless the entire amount being so exchanged for preferred stock can be added to the rate base. It is strongly urged that any privately owned water utility which has been extending extensively its construction by means of construction advances avoid building up too heavy a construction advance liability.

Bank loans, basically, are a temporary means of financing any construction program. No public utility can or should build up extensive bank loans without having first determined the manner in which they can and will refund these from a permanent source of capital.

Long-Term Debt

Many utilities place a great deal of emphasis on securing the lowest possible interest rate in developing their long-term borrowing. At times this may be a decided mistake, as the greater emphasis should be placed on securing the most genuinely advantageous general business terms. Creation of a long-term debt involves the careful development of a contractual relationship which must be worked out in sufficient detail to be satisfactory to both the borrower and lender.

Inexperienced borrowers sometimes misunderstand various loan terms, perhaps because they do not realize that the complexity of the agreement is necessary to provide sufficient flexibility to permit the utility to conduct a normal business over many years.

It is not practical to attempt to establish specific ratios indicating for each special circumstance how much long-term debt may be safely incurred. It is felt that the establishment of such ratios frequently oversimplifies the problem. Clearly well established utilities serving desirable areas, and with proved ability to raise additional equity when, as, and if needed, can prudently borrow larger amounts rela-

tively than small utilities whose management is less experienced in financing matters and whose service area is comparatively confined. Management should realize that creation of a debt imposes upon it a fixed charge not only for interest but ultimately for repayment of the amounts borrowed.

Excessive debt certainly lessens the general attractiveness of a utility's equity and makes it more difficult to interest additional investors in a company's preferred or common stock.

One of the most important considerations when selecting a program for permanent financing is the question of what effect this particular financing will have on the ability to raise additional funds. It is very poor policy to exhaust long-term credit in the hope that either additional funds will not be needed for many years or that when such additional funds are needed there will be no trouble in raising all that is required through the sale of equity securities. Under some circumstances equity capital is very difficult to find. Therefore, it seems wise to always keep some margin of borrowing capacity available for emergency use.

Interest Rates

Interest rates concern all business which must raise additional capital. Considering the large amount of capital needed by water and other utilities, this constitutes an even greater concern. In recent years, the cost of money has increased rather steadily. Most, at times, will forget that money even today is not historically high priced. Although interest rates are materially higher than those paid in the last 25 years, they are still generally low compared to those paid in the 1920's. Rates are also low in this country compared to almost any place else in the world. The change in rates should be viewed as a restoration of more normal interest rates. This increase has occurred as a result of extremely active business conditions which required heavy capital expenditures and borrowing by industry and individuals over the entire postwar period. The demand for capital in those years has been so heavy that the cost of money has been forced up to more normal levels after having been abnormally low during the prolonged period of depression and war ending in 1945.

California Regulations

The increasing cost of money has undoubtedly caused many to realize the necessity for rate adjustments. Certainly if utilities are to prosper in California, they must be permitted to earn a sufficiently high rate of return to enable them to attract capital on a fair basis. The belief, current in investment circles around the country, that regulations in California have been too strict is shared by the author.

Many utilities in other areas have been permitted to earn materially higher rates of return than do those in California. The area is certainly a growing one and the growth does not seem to be slackening. Unless investors believe that they will receive satisfactory earnings over a long period of time it is quite probable that they will tend to invest in other areas—not only from a geographical standpoint but also from an industry standpoint. Managers and owners of privately run water utilities should take every possible step to make sure that investors are well treated. This means that there is an obligation to the own-

ers to request rates sufficiently high to provide an adequate rate of return. If the return on capital invested in California utilities proves to be inadequate, relative to other states, it is going to be increasingly difficult to attract capital to the water industry. This ultimately would mean a deterioration of credit and of service to consumers. Earnings must be sufficient not only to attract capital but also to be fair to present investors.

There is no single continuing fair rate of return. The Public Utilities Commission in California certainly recognizes this fact and has generally been somewhat more lenient toward small utilities. This approach for the smaller utility is generally fair as the risks to the investor in placing his funds in smaller water companies are undoubtedly greater than in making investments in major, well established utilities.

Correction

In the paper "Review of the Jar Test" by A. P. Black, et al. (November 1957 JOURNAL, Vol. 49, pp. 1414–24), there was an editorial omission. The following sentence was inadvertently dropped from the text:

This investigation was supported by a research grant, RG-4516, from the National Institutes of Health, USPHS.

Relationships of Temperature and Air-Conditioning Water Use

Harris F. Seidel and John M. Carpenter

A contribution to the Journal by Harris F. Seidel, Supt., Water & Sewage Treatment, and John M. Carpenter, City Mgr., Ames, Iowa.

EARLY in 1955 the city council of Ames, Iowa, was made aware of the impact of air conditioning upon the local water utility. Water department personnel were asked to assemble information on the use of air conditioning locally and on current good practice in its control. With this background the city administration then prepared and adopted an ordinance limiting water use for this purpose. The following analysis derives from studies made at that time and subsequently.

Air-Conditioning Installations

At the time of the original survey, there were approximately 40 commercial and a dozen residential installations, totaling approximately 350 tons capacity. Although a few of the older commercial installations were of the straight-flow, noncondensing type, the majority were of the refrigeration type with no provision for water conservation.

The task of assembling data was simplified by the fact that both water and electric power use for almost all of these installations was separately metered. Water and electric meters are read together monthly in Ames; billing and collection for municipal water, sewage, and electric utilities is then handled through a central finance office.

In the few cases where metering was not completely separate, water and power consumption for other purposes was small and their use for air conditioning alone could be estimated quite accurately.

Water use in nonconserved air conditioning depends primarily on water temperature but is also affected by quality. For this reason, operating characteristics are different for every situation and those reported here are of general interest only.

Operating Data

Ames water is drawn from several shallow (120 ft) wells. Water is distributed at a temperature of 57°F in the business district in the summer months. Lime-soda softening and polyphosphate treatment produce a highly stable water of 80–90 ppm hardness.

At this temperature, water use per ton of cooling capacity was found to vary generally in the range 0.7–1.0 gpm while the condenser was operating, which is about 60 per cent of the time that the air-conditioning unit is in service.

Water use ranged from 30 to 45 gal/kwhr of power demand—independent of the size of the unit. This coincided roughly with a temperature rise of 35–40°F in the water passing

through the unit. In still other terms, the water use in refrigeration type units was 15,000–50,000 gal per ton per season. The straight-flow, noncondensing type conditioners were found to use roughly three times as much water—although, of course, no power was used except that for driving the blower.

Using a rough industry factor of 1.2 kwhr per ton of cooling capacity per condenser operating hour, the "condenser-hours" for each installation were calculated. Again, considering this to be roughly 60 per cent of the time the air conditioner was operating, the total operating hours (or "season") were calculated for each installation. These results ranged generally from 400 to 1,200 hr, except for restaurants, several of which had air-conditioning seasons of more than 2,000 hr.

Detailed operating data were tabulated for each installation and made available to local air-conditioning dealers and other interested persons. This information has been of considerable value in estimating operating costs for prospective customers. Indirectly, the study has also provided data of considerable value to the municipal water and electric utilities for demand and load growth studies.

Seasonal Use Patterns

Even the most casual review of the above operating data indicated that water demand for air conditioning responded to weather variations. After preliminary checks, it was decided to test for statistical correlation between water use and temperature patterns.

First, ten commercial air-conditioning installations were chosen with a range in type of business and size of unit which was considered to be representative of those in the business district at the time. Table 1 lists the installations and gives their water consumption during the four seasons prior to the original study in 1955 and for the two subsequent years. Two of the ten installations were changed radically in 1957, making continuation of the study in its original form impossible.

TABLE 1

Metered Water Use of Ten Representative Air-Conditioning Installations at Ames, Iowa

Installation	Location	Capacity tons*	Water Use During Air-Conditioning Season—1,000 cu ft						
			1951	1952	1953	1954	1955	1956	Average
A	Automobile showroom	5R	8.3	9.5	17.7	16.5	22.5	19.0	15.6
B	Shoe store	¾S	7.4	9.7	12.4	13.3	13.0	12.5	11.4
C	Lodge hall	11R†	28.4	53.2	41.0	50.3	66.8	44.0	47.3
D	Men's clothing store	5R	11.2	20.8	19.6	18.7	20.9	19.4	18.4
E	Drug store-luncheonette	3R	23.1	30.7	26.4	33.8	28.5	27.8	28.4
F	Restaurant	10R	90.1	99.6	151.9	122.5	125.0	118.0	117.8
G	Drug store	3R	4.5	7.0	9.1	13.4	16.9	14.7	10.9
H	Women's wear store	5R	10.2	12.9	14.0	14.1	13.1	12.6	12.8
I	Restaurant	11S	61.7	96.6	110.3	119.4	123.1	139.0	108.3
J	Department store	30R‡	132.6	201.0	229.1	200.0	195.0	225.3	197.2

* Key: R—Refrigeration or noncondensing type; S—Straight flow, noncondensing type.
† Combination of two 3-ton and one 5-ton unit.
‡ Combination of two 15-ton units.

Next, air-conditioning water use per season for each installation was expressed as a percentage of average use for a 6-year period (Table 2). The unweighted average of these ten values was then determined for each season. The latter were assumed to be representative of the year-to-year variation in water demand for air conditioning over the period under study. Note that, according to this rather arbitrary procedure, 1955 appeared to be about twice as rigorous an air-conditioning season in Ames as did 1951.

TABLE 2

Air-Conditioning Water Use Patterns for Ten Installations

Installation	Seasonal Water Use—Per Cent of 6-Year Average					
	1951	1952	1953	1954	1955	1956
A	53	61	114	106	144	122
B	65	85	109	117	114	110
C	60	113	87	106	141	93
D	60	113	106	102	114	105
E	81	108	93	119	101	98
F	76	85	129	104	106	100
G	40	64	83	123	155	135
H	80	101	109	110	102	98
I	57	89	101	111	114	128
J	67	102	116	102	99	114
A–J*	63.9	92.1	104.7	110.0	119.0	110.3

* Unweighted average.

The basic water use data were available for monthly meter-reading periods and, with these data, similar calculations were made for each installation and averaged for the ten, to yield a month-to-month pattern. Sample calculations for the 1953 season for installations A, J, and A–J inclusive are presented as part of Table 4. Water use by any one installation often varied appreciably from the general climate pattern but was balanced by demands of others in the group.

Seasonal Temperature Patterns

After establishing what it was believed were representative water use patterns, attention was turned to a study of climatic factors which might exhibit reasonably similar trends. This phase of the study was aided by the availability of continuously recorded temperature data for the 1953 and 1954 growing seasons in the Ames area, gathered by the Agronomy Department of Iowa State College.

TABLE 3

Summary of Temperature Data

Month*	Number of Degree-Days Above 65°F					
	1951†	1952	1953	1954	1955	1956
May	27	35	20	15	58	44
June	88	210	227	187	80	293
July	169	314	234	409‡	344	268
August	176	170	258	236	422‡	307
September	91	118	158	159	204	93
October	20	13	61	38	11	42
Total§	571	860	958	1,044	1,119	1,047

* Meter-reading month; average reading date is 25th of calendar month.
† Unusually cool and wet summer.
‡ Among the warmest summer months on record locally.
§ Percentages of 6-year average (933) of season totals are as follows: 1951, 61.2; 1952, 92.2; 1953, 102.7; 1954, 111.9; 1955, 119.9; and 1956, 112.2.

Statistical studies were made of a number of temperature relationships, the details of which are of only passing interest. First, data were accumulated on "cooling hours"—the actual hours during which air temperatures were above arbitrary bases such as 65°F, 70°F, and 75°F. The correlation between air-conditioning water use and cooling hours as thus defined was neither good nor bad but was closest at 65°F or 70°F.

Also investigated were "degree-hours"—the actual count of degrees

Fahrenheit times the number of hours above the bases of 65°F, 70°F, and so on. This yielded the poorest correlation with water use of the several temperature factors analyzed.

Next, a calculation of "degree-days" above various temperatures was made. This is an exact counterpart of the degree-days factor so familiar to the heating industry, which is based on variance of the mean daily temperature from a given base such as 65°F.*

it was realized that local mean daily temperatures of 65°F would correspond to daytime maximums just becoming uncomfortable and that air-conditioning requirements would increase with temperatures yielding degree-days above a 65°F base.

Degree-days above 65°F for the 6-year period 1951–56 inclusive are summarized in Table 3. A sample calculation of the temperature pattern for the 1953 season is shown as part of Table 4. Month-to-month temperature and water use patterns for the other years studied are presented in Table 5.

TABLE 4

Comparison of Air-Conditioning Water Use for Two Installations and Temperature During 1953 Season

Month*	Water Use					Degree Days‡	
	Installation A		Installation J		A–J†		
	1,000 cu ft	Per Cent of Season Total	1,000 cu ft	Per Cent of Season Total	Per Cent of Season Total	Number	Per Cent of Season Total
May	—	—	16.12	7	1.7	20	2.1
June	3.41	19	51.27	22	25.1	227	23.7
July	5.37	30	49.86	22	23.8	234	24.4
Aug.	5.80	33	55.99	24	28.9	258	26.9
Sept.	2.65	15	31.52	14	14.5	158	16.5
Oct.	0.51	3	24.37	11	6.0	61	6.4
Season total	17.74	100	229.13	100	100.0	958	100.0

* Meter-reading month; average reading date is 25th of calendar month.
† Refers to all installations listed in Tables 1 and 2.
‡ Base of 65° F.

This appeared to yield a poor correlation when calculated for base temperatures of 75°F or 80°F, where it might be anticipated that air conditioning would become important. At 70°F this correlation was considerably improved, and at 65°F it was surprisingly good. This was rather puzzling until

The very striking correlation between air-conditioning water use and this particular climate factor, year after year, is shown in Fig. 1 and 2. The length of the study period would seem to rule out the play of coincidence or chance in this relationship.

Humidity, in the range recorded, appeared to have little or no effect.

* For example, using a base of 65°F, with a recorded maximum temperature of 95°F and a minimum of 75°F, the mean temperature would be 85°F. This would give cooling degree-days of 85°F–65°F or 20 degree-days for that day.

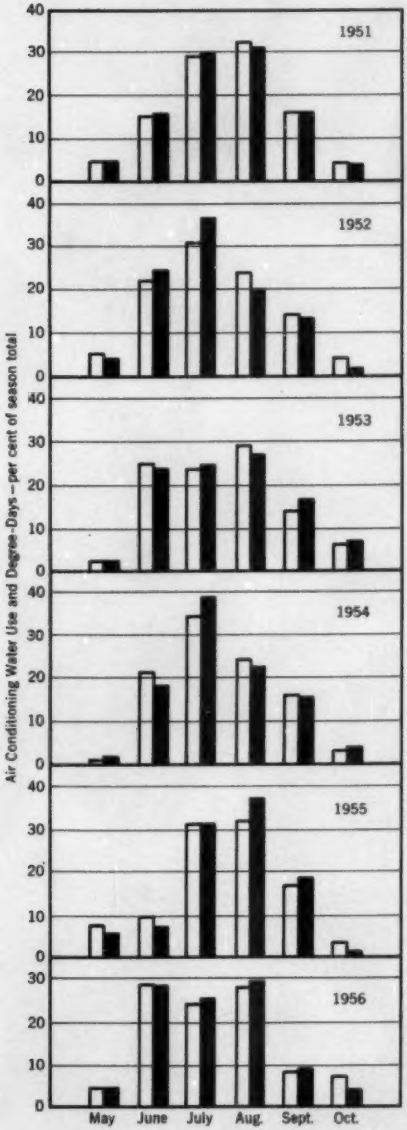

Fig. 1. Monthly Water Use and Temperature Relationships, 1951–56

Data shown are for local conditions at Ames, Iowa, only. Solid bars show degree-days above base of 65°F; open bars indicate air-conditioning water use.

Relative humidities noted during these seasons, as measured at 12:30 PM and 6:30 PM, were generally in the 50's during hot, dry periods and in the 60's during cool, wet periods.

The obvious qualification must be added that the correlation described was determined for Ames, Iowa, and for the water supply and weather conditions found locally; nothing more is implied. If such a correlation with degree-days has value, however, its advantages include ready availability of the temperature data needed and consequent ease of calculation.

Residential Installations

At the time of the original study, valid operating data were available for only a few of the dozen residential installations then in service—most of 2- or 3-ton capacity. These and subsequent records, however, indicated the water use *for air conditioning alone* to average generally 600–1,200 gpd per installation during the peak summer month.

It is not easy to grasp the significance of these figures. One implication is that the water use rate in a

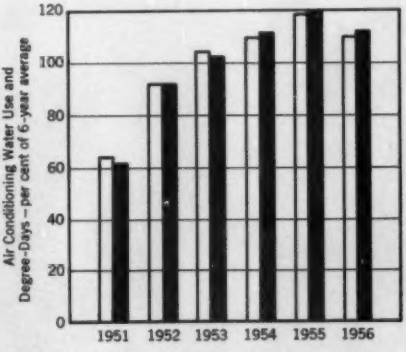

Fig. 2. Annual Air-Conditioning Water Use-Temperature Relationships

Solid bars indicate degree-days; open, air-conditioning water use.

TABLE 5

Comparison of Air-Conditioning Water Use and Temperature for 6-Year Period *

Month†	Per Cent of Season Total									
	1951		1952		1954		1955		1956	
	Water Use	Degree Days	Water Use	Degree Days	Water Use	Degree Days	Water Use	Degree Days	Water Use	Degree* Days
May	4.6	4.7	5.1	4.1	1.2	1.5	7.5	5.2	4.6	4.2
June	15.3	15.4	22.6	24.4	20.5	17.9	9.4	7.2	28.4	28.0
July	28.3	29.6	30.5	36.5	35.1	39.2	30.8	30.7	23.9	25.6
August	32.1	30.9	24.4	19.8	24.4	22.6	32.2	37.7	27.9	29.3
September	15.7	15.9	13.7	13.7	16.0	15.2	17.4	18.2	8.3	8.9
October	4.0	3.5	3.7	1.5	2.8	3.6	2.7	1.0	6.9	4.0

* Data for 1953 are included in Table 4.
† Meter-reading month; average reading date is 25th of calendar month.

neighborhood saturated with non-conserved air conditioning would triple or quadruple during the warmest summer months, simply on the basis of total monthly use. It is hard to grasp the possible effect in terms of short-term peak loads, for there simply is not that much water, nor are distribution facilities capable of delivering it to the point of use.

Adoption of Ordinance

Calculations for the 1954 season showed that air-conditioning water use in Ames was only 2.5 per cent of total annual water sales and probably did not exceed 8 per cent of the rate of use during peak demand periods. These small proportions were in themselves hardly of major concern. Nevertheless, the city council was concerned with at least three facets of this problem. First, air conditioning was obviously in its ascendancy, and its effect on the water utility could only grow. Second, if residential installations mushroomed as anticipated by the air-conditioning industry, the demands on source, plant, and distribution system would become insupportable. Finally, the air-conditioning season in Ames comprised possibly 10 per cent of the year and facilities provided to serve these unusual demands would be idle and unproductive the remaining 90 per cent of the time.

Another important factor was the tone of the objections to any type of control raised by owners of existing installations. The council quickly concluded that only more severe water demands and a larger group of objectors could result from delay or postponement of control.

Consequently, the city administration prepared and adopted during the 1955 season a rather restrictive ordinance, effective the following year. Based upon the model ordinance prepared by the AWWA (*1*), the Ames ordinance states that new installations of 2 tons and over in capacity must provide water conservation, limiting the demand rate to 0.25 gpm per ton. The ordinance is not retroactive—that is, it does not apply to existing installations during their normal service lives, but will apply to existing units at the time of alteration or replacement of components.

With the background of operating and cost data furnished them and after several conferences with city officials, the local air-conditioning people ac-

cepted the new ordinance with the realization that it would insure more reliable and more economical water service for the community as a whole.

Summary

A study of water use for air conditioning in Ames, Iowa, yielded detailed operating records which focused attention on the impact of this demand on the local water utility.

After review of the problem by city officials with local business men and industry representatives, a restrictive ordinance was prepared and adopted. Complaint or comment on the ordinance since passage has been virtually nil.

From this study a surprisingly close correlation was derived between climate—in terms of degree-days above 65°F—and water use by a group of representative, local air-conditioning installations.

Reference

1. Committee Report. Regulation of Air Conditioning and Other Refrigeration. *Jour. AWWA*, **42**:1111 (Dec. 1950).

Effect of 1952–56 Drought on Iowa Impounded Water Supplies

E. Robert Baumann and John L. Cleasby

A paper presented on Oct. 17, 1957, at the Iowa Section Meeting, Des Moines, Iowa, by E. Robert Baumann, Prof., and John L. Cleasby, Asst. Prof., both of the Dept. of Civil Eng., Iowa State College, Ames, Iowa.

SINCE September 1952, Iowa has had the most serious drought in its recorded weather bureau history. The lack of precipitation has been fairly widespread in the midwestern United States and has been particularly acute in southern Iowa. As a result of the extended drought, sixteen of the 21 cities in southern Iowa which use impounding reservoirs as a source of water supply experienced serious water shortages (Fig. 1). In some cities supplies were inadequate, and water had to be brought in, often from a distance. In other communities the use of water was curtailed to conserve the dwindling supply. Six of the cities were fortunate in having auxiliary sources which could be used.

Although the primary cause of water shortages has been the extended period of drought, many reservoir inadequacies were due to improper hydrologic design. The adequacy of an impounded source to meet a specified demand is controlled by two closely related factors, the reservoir volume and the contributing drainage area. To be fully adequate as a water supply source, the reservoir volume must be large enough to supply water over the most severe drought that can reasonably be expected.

In addition, the drainage area must be large enough to fill the reservoir and to contribute enough water to maintain an adequate supply during the design period drought. Overdesign of one factor will partially compensate for a deficiency in the other. Many of the impounded water supplies of Iowa are inadequate in one or both of these factors.

Therefore, the purpose of the study was:

1. To determine the rainfall deficiency for the 1952–56 drought as compared with that of previous droughts
2. To determine the runoff deficiency for the 1952–56 drought
3. To determine the effect of these deficiencies on the adequacy of Iowa municipal impounded supplies by means of a questionnaire survey
4. To determine in a qualitative manner the design weaknesses which caused the inadequacies
5. To propose information which may prove helpful in preventing recurrence of these difficulties.

The scope of this study was limited to southern Iowa, particularly to the last two tiers of counties. It was further limited to a qualitative evaluation of the impounded supplies, because of the lack of small-basin runoff records

Fig. 1. Empty Iowa Reservoir

This picture, taken in 1955, shows the effect of the prolonged drought discussed by the authors.

in southern Iowa. No specific design recommendations have been made in this study.

Other Studies

Quantitative studies of reservoir design have recently been made by Rosebraugh (*1*) for the eastern section of Kansas and by Hudson and Roberts (*2*) for Illinois. The importance of providing adequate reservoir volume and an adequate drainage area was emphasized in both reports. Rosebaugh suggested a minimum of 125,000 gal storage and 2.25 drainage acres per person for areas in eastern Kansas where average annual rainfall is 40 in.

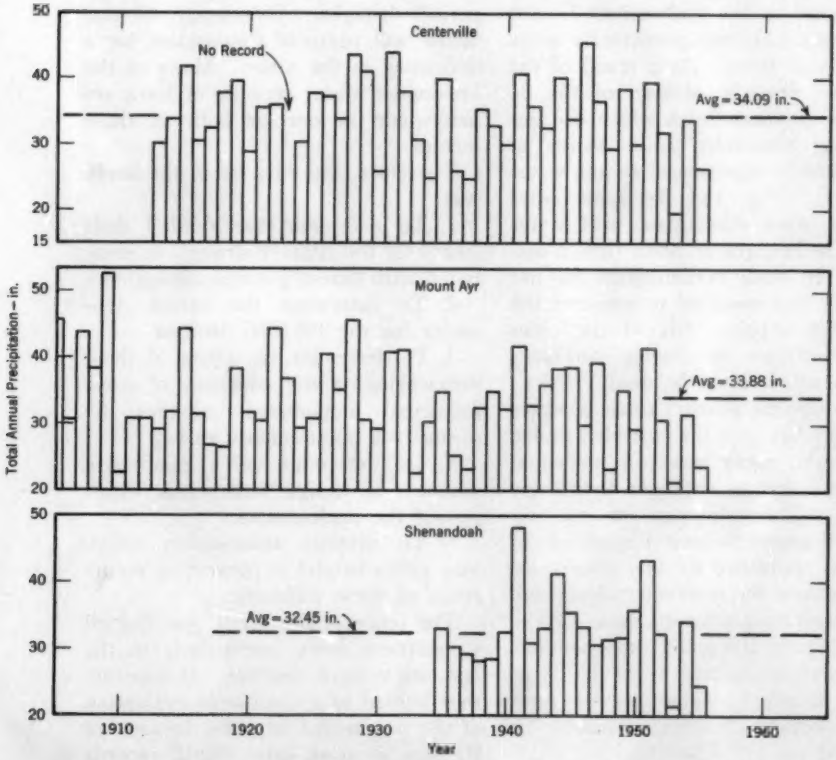

Fig. 2. Variations in Annual Precipitation for Three Iowa Communities

For areas with less rainfall and consequently less runoff, he proposes proportionately more storage and area.

In 1935, Wieters (*3*) evaluated the effect of the 1933–34 drought on Iowa public water supplies. Quantitative design recommendations were not possible due to lack of adequate runoff data, but the principal cause of failure of impounded supplies was determined to be insufficient storage, with drainage area having little effect. Wieters recommended a program for increasing the available hydrologic data for future use in improving the design of impounding reservoirs. In 1936, Waterman et al. (*4*) presented some fundamental design considerations for the design of impounding reservoirs based on the study by Wieters and a study of rainfall runoff data from Ralston Creek, Iowa. It is interesting to note that most of the communities which experienced failures of impounded supplies in the 1933–34 drought also experienced failure or shortages in the 1952–56 drought. Today, there is still a lack of adequate hydrologic data, which Wieters indicated were needed in 1935. Nixon and Schwab (*5*) have proposed a method for predicting small watershed yield based on watershed characteristics in southern Iowa.*

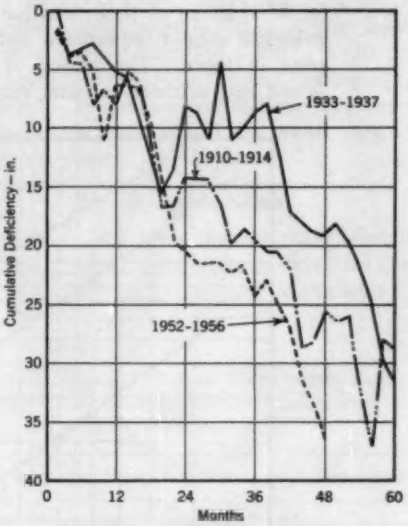

Fig. 3. Rainfall Deficiencies During Droughts at Mount Ayr, Iowa

Curves show that the cumulative deficiencies during the 1952–56 drought were more severe than those during comparable periods in the other two droughts plotted.

Extent of Drought

The normal annual precipitation in the area of southern Iowa under consideration is about 35 in., but the precipitation which may occur in any year may vary greatly from the average. Annual variations in precipitation for the entire period of record for three Iowa stations are shown in Fig. 2. These three stations—Mount Ayr, Centerville, and Shenandoah—were se-

* Other studies have been made on the relationships between water storage development and runoff data, beginning in 1914 with Allen Hazen (*6*), who based his analyses primarily on watersheds of New England and New York. Boyce, et al. (*7*) discussed the problem on the basis of data from eastern and midwestern waters, and Smallwood, Wyndham, and Cole (*8*) have presented material from a North Carolina river.

Including this article and the references the authors have given (*1–5*), it would appear that there is quite a bit of material published from a regional point of view. This leads to the question of whether it would be possible or feasible—considering differences in climate, rainfall, and local conditions—for someone or some group to analyze these sources of material, along with others, and present an overall picture of the water storage situation as it affects the country at large. Such a study, if undertaken, would make an interesting and worthwhile project.—Ed.

lected because of their location in the principal area of impounded water supplies in Iowa. The period of record is not long at these stations, except for that at Mount Ayr, which has kept records for 60 years. Few stations in southern Iowa have records for more than 20 years.

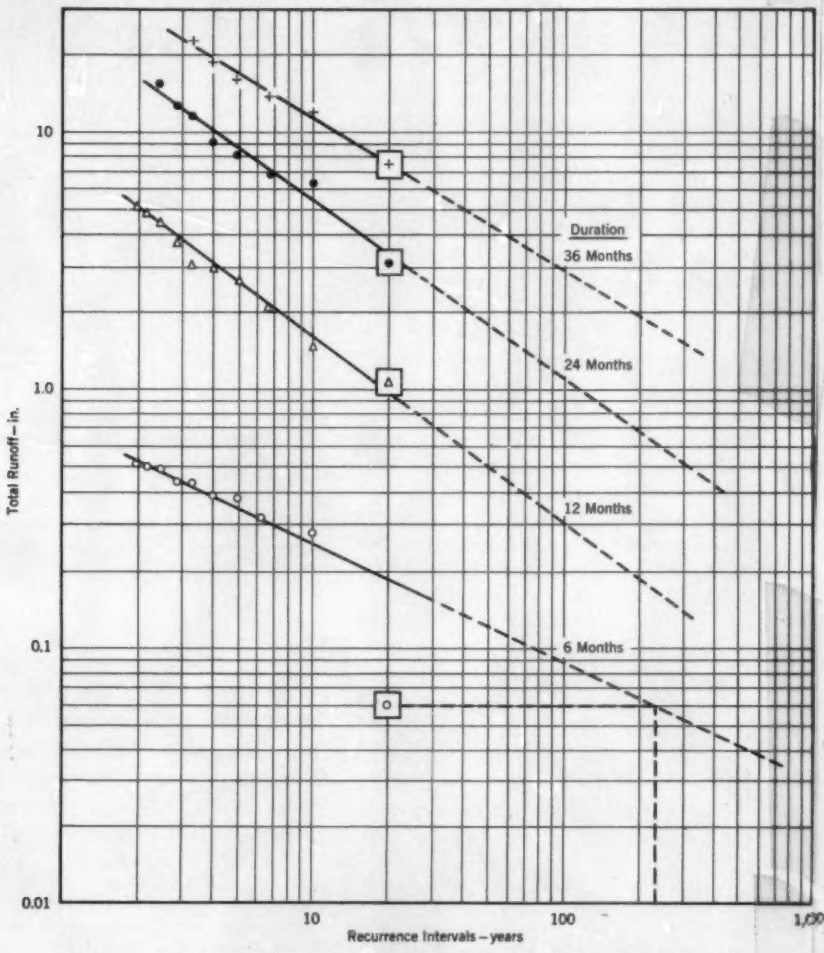

Fig. 4. Recurrence Intervals for Low-Flow Periods

Symbols are plotted from various low-flow periods of indicated duration, from records of the Thompson River, at Davis City, Iowa. Symbols inside squares indicate the 1952–56 drought, where the 6-month total falls far below its appropriate curve. In fact, it can be seen by the dashed lines that a 6-month period as severe as this could be expected to occur about once every 250 years.

The rainfall deficiencies plotted in Fig. 3 for Mount Ayr show the 1952–56 drought as compared with previous drought periods. From these figures it is evident that the deficiencies experienced in the 1952–56 drought were more severe than any previous drought on record, amounting to 36.5 in. in the critical 48-month period.

Since water demands are greatest in the summer months, the seasonal variation of this deficiency occurrence was studied for the three stations. This is shown in Table 1, which compares the normal monthly rainfall distribution with the average monthly distribution during the four driest years on record. This table shows that most of the deficiencies occur from April to September.

This distribution of rainfall compounds the deficiency problem. Lack of rainfall in the summer months results in unusually dry soil moisture conditions, which lead to proportionately more infiltration and less runoff when rains do occur. A 50 per cent deficiency in rainfall occurring mainly in the summer months may reduce the normal runoff by more than 50 per cent.

Runoff Deficiencies

To determine the runoff deficiencies, records covering an extended period of stream flow should be available for several streams in the area concerned. Such records are usually not available, particularly for basins of the size normally used for municipal supplies. Short-period records were available for the Thompson River at Davis City, Iowa; and to obtain at least a spot indication of runoff deficiencies, these records have been analyzed for low-flow frequencies and durations. The Thompson River was selected because

TABLE 1

Monthly Precipitation During Normal Years as Compared With Drought Years

Month	Avg. Monthly Precipitation—*in.*					
	Centerville		Mount Ayr		Shenandoah	
	Normal	Drought*	Normal	Drought*	Normal	Drought†
January	1.11	1.57	1.10	1.21	0.85	0.92
February	1.30	1.60	1.19	1.31	1.10	1.59
March	1.86	1.84	1.87	1.30	1.55	1.49
April	3.20	2.78	3.09	2.62	2.60	2.41
May	4.30	2.38	4.19	2.47	4.10	2.26
June	4.83	3.12	5.21	4.21	5.00	6.13
July	3.60	2.40	3.65	2.34	3.70	1.95
August	3.73	1.87	4.00	1.64	4.10	2.28
September	4.15	2.40	3.89	3.01	4.05	2.46
October	1.66	1.59	2.69	1.58	2.60	0.81
November	1.99	1.94	1.83	2.49	1.75	0.71
December	1.30	0.66	1.15	0.46	1.05	0.66
Totals	33.03	24.15	33.86	24.64	32.45	23.67

* Average of four driest years: 1934, 1937, 1953, and 1955.
† Average of two driest years: 1953 and 1955.

on it is the only gaging station in south central Iowa. With a drainage basin of 704 sq mi, it is larger than most municipal basins, but it is the best available station. Records are available for the 18-year period from 1918–25 and 1941–55. This is typical of the lengths of records available on Iowa streams and emphasizes the need of more basic hydrologic data.

USGS water supply papers for the Thompson River are tabulated in Table 2. To obtain the recurrence interval data to be used in plotting each of the curves of various durations in Fig. 4, it was necessary to make a running tabulation of the total runoff for the duration period concerned. A typical tabulation for the 6-month duration curve is included in Table 3.

TABLE 2

Runoff of Thompson River at Davis City, Iowa

Year	Total Runoff—*in.*											
	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1918					0.55	0.92	0.07	0.06	0.01	0.08	0.13	0.09
1919	0.10	0.25	1.42	1.86	2.21	3.94	0.13	0.13	0.03	0.30	0.83	0.04
1920	0.03	0.86	1.90	2.77	1.34	0.66	0.63	0.20	1.38	0.06	0.06	0.07
1921	0.04	0.22	0.23	0.52	0.74	0.72	0.04	0.06	0.30	0.08	0.06	0.02
1922	0	0	0.33	0.21	0.83	0.15	2.60	0.86	0.60	0.15	1.57	0.13
1923	0	0	1.29	0.52	0.28	0.24	0.25	0.26	0.19	0.16	0.05	0.05
1924	0	0	1.26	0.22	0.72	3.39	0.39	0.20	0.31	0.05	0.04	0
1925	0	0.18	0.16	0.10	0.04	0.56						
1941							0.02	0.02	1.30	2.32	1.93	1.02
1942	1.53	0.56	1.14	0.31	1.12	0.86	0.48	0.13	0.18	0.15	0.15	0.30
1943	0.14	0.46	0.30	0.16	0.96	2.07	0.40	0.28	0.09	0.03	0.04	0.02
1944	0.04	0.10	0.55	2.61	2.20	1.00	0.17	0.60	0.29	0.18	0.12	0.32
1945	0.14	1.01	2.53	2.47	2.08	0.95	0.26	0.08	0.18	0.15	0.06	0.12
1946	1.82	0.53	1.31	0.59	0.91	1.18	0.13	0.29	0.18	0.37	0.26	0.10
1947	0.11	0.17	0.79	3.56	0.96	7.55	0.43	0.05	0.02	0.04	0.09	0.22
1948	0.04	0.31	1.52	0.36	0.33	0.04	0.11	0.07	0.02	0.01	0.05	0.02
1949	0.17	0.77	0.94	0.36	0.14	1.07	0.37	0.16	0.10	0.04	0.03	0.04
1950	0.01	0.82	0.67	0.10	1.53	1.61	0.26	0.39	0.04	0.02	0.02	0.01
1951	0.01	1.14	1.40	2.14	4.27	2.03	0.92	0.32	0.25	0.74	0.55	0.09
1952	0.38	0.22	2.63	0.84	1.01	0.99	0.04	0.05	0.01	0.01	0.32	0.08
1953	0.09	0.67	0.87	0.87	0.45	0.59	0.08	0.08	0.02	0.01	0.01	0.01
1954	0	0.01	0.02	0.07	0.18	0.77	0.02	0.06	0.01	0.28	0.01	0.01
1955	0.04	0.55	0.60	0.21	0.06	0.03	0.31	0.02	0.01			

The purpose of this phase of the study was to determine the relationship between the frequency of occurrence (recurrence interval) of drought periods of varying duration and severity. The results are shown in Fig. 4, which was plotted from the method described by Hudson and Roberts (*2*). The original runoff data obtained from the

To obtain the values in this table, each 6-month period was totaled and tabulated in the middle of the period. For example, the figure of 1.69 under August 1918 (Table 3) is the total of the May–October figures in Table 2.

When the running totals have been tabulated for the entire period of record, the table is scanned for the lowest

6-month total, which is given the order *1*. Then the second lowest 6-month total is found and given the order *2*, and so on. As each is found and numbered, the months used in obtaining that total are crossed out so that this 6-month period is not included in more than one occurrence. The recurrence interval is then obtained by dividing the total years of record by the order number, which also indicates the number of times runoffs of that amount or less have occurred. For example, four occurrences (Order *4*) in 20 years of records would indicate a recurrence interval of $20 \div 4 = 5$ years.

The severity of the 1952–56 drought is clearly evident from the recurrence interval curves in Fig. 4. For all durations, the 1952–56 drought resulted in the lowest total runoff, which would indicate that this drought was the most severe drought on record for the Thompson River basin. Furthermore, the 6-month point for the 1952–56 drought falls far below the appropriate curve. This would indicate that the recurrence interval of this point is probably greater than 20 years. In Fig. 4, movement of the 6-month point for the 1952–56 drought horizontally until it intersects the 6-month line indicates an actual recurrence interval of about 250 years, rather than 20 years.

It is evident that if sufficient data were available to develop similar curves

TABLE 3

Six-Month Running Total, Runoff of Thompson River*

Year	Six-Month Running Total—*in.*											
	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1918								1.69	1.27	0.44[7]	0.47	0.66
1919	2.07	3.85	5.93	9.78	9.81	9.69	8.30	6.44	5.36	1.46	1.36	2.09
1920	3.96	6.43	6.94	7.56	8.16	7.50	6.98	4.27	2.99	2.40	1.81	1.83
1921	0.68	1.14	1.82	2.47	2.47	2.31	2.38	1.94	1.26	0.56	0.52	0.46[8]
1922	0.49	0.62	1.39	1.52	4.12	4.98	5.25	5.19	5.93	5.91	3.31	2.45
1923	3.14	3.51	2.22	2.33	2.58	2.84	1.74	1.38	1.15	0.96	0.71	0.45
1924	4.66	1.58	2.25	5.60	5.98	5.98	4.84	5.06	4.58	0.98	0.60	0.58
1925	0.43[6]	0.48	0.48	1.04								
1941										6.61	8.12	8.66
1942	8.93	6.49	5.68	5.52	4.47	4.04	3.08	2.77	1.95	1.39	1.05	1.38
1943	1.50	1.51	2.32	4.09	4.35	4.17	3.96	3.80	2.91	0.86	0.50	0.32[3]
1944	0.78	3.36	5.52	5.60	6.63	7.13	6.87	4.44	2.36	1.68	1.65	2.06
1945	4.30	6.59	8.55	9.18	9.30	8.36	6.02	3.70	1.68	0.85	2.41	2.86
1946	3.99	4.43	5.28	6.34	4.62	4.41	3.28	3.06	2.41	1.33	1.31	1.19
1947	1.80	4.49	5.69	13.14	13.46	13.34	12.57	9.05	8.18	0.85	0.46	0.72
1948	2.22	2.54	2.78	2.60	2.67	2.43	0.93	0.58	0.30	0.28[2]	0.34	1.04
1949	1.96	2.31	2.40	3.45	3.65	3.04	2.20	1.88	1.77	0.74	0.38[4]	1.04
1950	1.61	1.67	3.17	4.74	4.99	4.17	3.93	3.85	2.34	0.74	0.50[9]	1.24
1951	2.60	4.72	8.97	10.99	11.90	10.76	9.93	8.53	4.81	2.87	2.33	2.23
1952	4.11	4.71	5.17	6.09	5.73	5.56	2.94	2.11	1.42	0.51[10]	0.56	1.18
1953	2.05	2.90	3.03	2.67	3.53	2.94	2.09	1.23	0.79	0.21	0.13	0.06
1954	0.06[1]	0.12	0.29	1.05	1.07	1.12	1.11	1.32	1.15	0.39[5]	0.41	0.90
1955	1.49	1.42	1.47	1.49	1.75	1.24	0.64					

* Drought order is indicated by the superscript numerals at appropriate figures, as follows: 1, 0.06 in.; 2, 0.28 in.; 3, 0.32 in.; 4, 0.38 in.; 5, 0.39 in.; 6, 0.43 in.; 7, 0.44 in.; 8, 046 in.; 9, 0.50 in.; and 10, 0.51 in.

TABLE 4—*Survey Data for Impounded Water Supplies in Southern Iowa*

Town	Population			Water Consumption			Reservoir Data				Watershed Area *acres*	Storage per Capita *1,000 gal.*	Watershed Area per Capita *acres*
	1930	1950	Change %	1935 *mil gal/yr*	1955 *mil gal/yr*	Change %	Pond Area *acres*	Original Capacity *mil gal*	Avg. Depth* *ft*	Year Built			
Afton	1,013	936	− 7	—	25	—	18	70	12	1935	179	75	0.2
Albia	4,425	4,838	+ 9	11	20	+ 90	48	370	23	—	805	77	0.2
Allerton	784	761	− 3	—	26	—	110	187	5	1913	3,700	246	4.9
Bedford	2,000	2,000	0	30	65	+116	125	250	7	1936	3,937	125	2.0
Bloomfield	2,732	2,688	− 2	21	64	+205	88	319	11	1937	1,320	119	0.5
Centerville	8,147	7,625	− 7	170	240	+ 41	19	100	16	1914	770		
							153	640	13	1922	1,770	97	0.3
Chariton	5,365	5,320	− 1	137	150	+ 10	78	240	10	1915	800		
							72	395	17	1915	885	181	1.2
							110	330	9	1940	4,500		
Corning	2,026	2,104	+ 4	37	56	+ 50	20	80	12	1917	600		
							61	170	9	1943	2,100	119	1.3
Corydon	1,768	1,870	+ 6	28	43	+ 50	38	70	6	1917	1,920	37	1.0
Creston	8,615	8,317	− 3	310	240	− 23	348	470	4	1891	9,850		
							47	66	4	—	2,500	268	1.5
							480	1,700	11	—	—		
Fairfield	6,619	7,299	+10	190	225	+ 18	30	70	7	1885	255		
							40	170	13	1911	512	67	0.4
							65	255	12	1954	2,300		
Greenfield	1,837	2,102	+14	—	48	—	25	83	10	1936	600	40	0.3
Humeston	924	750	−19	22	18	− 18	14	45	10	1914	650	60	0.9
Lamoni	1,739	2,196	+26	15	43	+187	30	42	4	1908	430	19	0.2
							95	196	6	1956	2,700	†	
Lenox	1,717	1,171	−32	17	20	+ 18	14	34	7	1912	90		
							35	142	12	1923	750	150	0.7
Montezuma	1,257	1,460	+16	—	40	—	110	250*	7	—	2,500	171	1.7
Mt. Ayr	1,704	1,793	+ 5	—	88	—	10	23	7	1914	640		
							95	286	9	1942	2,700	172	1.8
Osceola	2,871	3,422	+20	30	100	+234	55	155	9	1934	10,000		
							48	95	6	1945	400	73	3.0
							40	71	5	1956	252	†	
Seymour	1,571	1,223	−23	15	28	+ 87	10	30	9	1908	410	25	0.3
Tabor	1,048	869	−17	13	17	+ 31	2	4	6	1955	1,400	5	1.6
Winterset	3,500	3,570	+ 2	—	55	—	92	235	8	1939	10,000	66	2.8

* Calculated. † Not included, since reservoir was not built until after drought period.

for several streams with smaller drainage areas and longer records, these could be used to predict with reasonable accuracy the minimum total runoff that might occur in a given period of time. With this information at hand, a rational approach could be made in the selection of the reservoir volume and drainage area required to carry the city demand over an extended drought of a particular recurrence interval.

The runoff data for the Thompson River are not representative of the runoff to be expected from the small municipal watersheds, which are generally less than 20 sq mi. These small watersheds may produce zero runoff for periods in excess of 12 months, due to locally severe drought conditions.

Results of Survey

To determine the extent of and the possible reasons for the water supply shortages which resulted from the 1952–56 drought, a questionnaire was mailed to all communities with impounded supplies in Iowa. The results of the questionnaire survey are given in Table 4.

One cause of water shortage in many states has been the rapid increase in population. Although the rate of population increase in southern Iowa is not great, the seriousness of the water shortages has been compounded by a rapid increase in water consumption per capita. For example, Bedford and Bloomfield, whose populations have remained nearly constant since 1930, have had increases in consumption of 216 per cent and 208 per cent respectively. In general, the per capita consumption for all towns studied is still below the current average for the whole country.

Before considering the number and extent of inadequacies of impounded water supplies, it was necessary to define the basis upon which these critical water supplies should be judged. A supply was considered to be *inadequate* only if more elaborate measures were undertaken than the restriction of water use which would normally be exercised by a community faced with a water shortage. A supply was considered to be inadequate only if it was necessary to supplement the impounded water supply source to maintain the health and sanitation of the community. The supplementary water supply sources varied, depending both upon their availability and the local severity of the drought. Some communities had deep wells or had drilled new wells which were used in the emergency period. Other communities not so fortunate were forced to haul water by train or tank truck from other sources. Of the 21 communities surveyed, eleven had inadequate supplies at some time during the 1952–56 drought.

In addition, five communities had periods in which the supply was *limited*—that is, requiring only the restriction of use normally exercised during a water shortage.

The basic cause of water supply inadequacy was the extended period of below-normal precipitation. If the impounded supplies had been adequately designed to carry the communities over a drought of this severity, however, there should have been fewer shortages. A study of Table 4 illustrates the important reasons for shortages, stemming from increased consumption or inadequate reservoir volumes or watershed areas.

Communities which had inadequate supplies generally were those with the greatest increase in consumption after construction of the reservoir facilities. Those with limited supplies were also

characterized by increased consumption, and those with adequate supplies were characterized by decreased or only slightly increased consumption. The consumption change information is summarized in Table 5.

Although the data are not complete enough to draw quantitative conclu-

TABLE 5

Summary of Factors Affecting Water Supply Adequacy

1	2	3	4	5	6	7
Town	Change in Consumption *per cent*	Watershed		Reservoir Volume		Total of 4 and 6
		Acres per Capita	Percentage of Kansas Standards*	Per Capita *1,000 gal*	Percentage of Kansas Standards*	
Inadequate Supplies						
Afton	—	0.2	7	75	53	60
Albia	+90	0.2	7	77	54	61
Bedford	+116	2.0	77	125	88	165
Centerville	+ 41	0.3	12	97	68	80
Fairfield	—	0.4	15	67	47	62
Lamoni	+187	0.2	8	19	13	21
Osceola	+234	3.0	115	73	51	166
Seymour	+ 87	0.3	12	25	18	30
Tabor	+ 31	1.6	62	5	3	65
Winterset	—	2.8	108	66	47	155
Greenfield	—	0.3	12	40	28	40
Average	+112	1.0	39	61	43	82
Limited Supplies						
Bloomfield	+205	0.5	19	119	84	103
Corning	+ 50	1.3	50	119	84	134
Corydon	+ 50	1.0	39	37	26	65
Humeston	− 18	0.9	35	60	42	77
Lenox	+ 18	0.7	27	150	105	132
Average	+ 61	0.9	34	97	68	102
Adequate Supplies						
Allerton	—	4.9	189	246	173	362
Chariton	+ 10	1.2	46	181	128	172
Creston	− 23	1.5	58	268	189	247
Mt. Ayr	—	1.8	69	172	121	190
Montezuma	—	1.7	65	171	120	185
Average	− 6%	2.2	85	209	147	232
Average omitting Allerton		1.6	62	198	139	201

* As recommended by Rosebraugh (*1*). Communities with a total of 170 per cent of the Kansas recommendations had adequate supplies.

sions, a study of Table 5 indicates that a lack of adequate reservoir volume, or a lack of sufficient watershed area, or both, may result in inadequate supply. A shallow reservoir will increase evaporation loss and cause failure of a system which might be adequate in all other respects. A weakness in Iowa impounding reservoirs is their shallow depth. This results in very high evaporation losses in the hot, dry, summer drought periods.

To obtain an indication of the adequacy of the reservoirs on these two points, those in Iowa were compared with the minimum standards recommended for Kansas (*1*). The adjusted minimum drainage area and reservoir volume as recommended for Kansas were found to be 2.6 acres per capita and 142,000 gal per capita respectively, for an average rainfall of 35 in. in Iowa. Table 5 indicates that none of the communities with inadequate supplies had adequate reservoir volume and drainage area based on the recommended minimum standards. Most of those with inadequate supplies were low in both watershed area and reservoir volume. Some were adequate in one value but low in the other. None of those with limited supplies satisfied both criteria.

It would appear that reservoir volume is the more important of the two criteria. Osceola and Winterset, which satisfied watershed area requirements but did not satisfy reservoir volume requirements, had inadequate supplies. On the other hand, four of the five communities with adequate supplies had inadequate watershed areas, but had reservoir volumes considerably above those of the Kansas requirements.

Southern Iowa is gently rolling and does not lend itself to ideal impounded reservoir sites. Desirable characteristics for the reservoir, such as a narrow neck to reduce the dam length and steep side slopes to minimize surface area and increase average depth, are seldom available. An otherwise suitable reservoir site often cannot be used because of an inadequate drainage area.

Conclusions

A qualitative survey of the 1952–56 drought has been presented along with an evaluation of the effects of this drought. The following conclusions may be drawn.

1. The 1952–56 drought was the most severe drought on record in southern Iowa.

2. This drought resulted in the longest period of low runoff on record for the Thompson River drainage basin above Davis City, Iowa. As an example of the drought severity, in one 6-month period during the drought, the total runoff was only 0.06 inch. The recurrence interval for such a 6-month period is predicted to be about 250 years.

3. The need for more hydrologic data is further emphasized, particularly runoff data for small watersheds. Such data, if available, would lead to more concrete quantitative recommendations of design for future reservoirs.

4. From an evaluation of existing supplies to determine whether they meet the suggested minimum Kansas requirements (*1*), it was found: that no supply which satisfied both of the requirements was inadequate; that most supplies which were very high in reservoir volume per capita but somewhat low in watershed area were adequate; and that all supplies which did not satisfy either of the requirements were either limited, requiring restrictions, or inadequate, requiring supplementation.

It therefore seems plausible that the requirements could be used as a basis of design for Iowa impounded supplies. These requirements for an area in Iowa with an average annual rainfall of 35 in. are: 142,000 gal reservoir volume per capita, and 2.6 acres of watershed per capita. Provision of an excess reservoir volume will offset a watershed area which does not completely satisfy this requirement.

Acknowledgments

Grateful acknowledgments are made to J. A. Power of the Iowa Natural Resources Council and to the operators who cooperated by furnishing the data tabulated in the paper.

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Selection of Meters and Services

Panel Discussion

A panel discussion presented on Mar. 19, 1957, at the Southeastern Section Meeting, Charleston, S.C.

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SELECTION of the proper meter size is one of the most complicated and difficult tasks which confront water works operators, many of whom are finding that what was good enough in grandfather's time is not good enough today. The selection of the proper type meter is relatively easy, but determination of the proper size requires careful calculation together with experience and practical knowledge.

Service Line Size

The whole problem of service and meter sizes, according to some waterworks operators, is caused directly by the inadequate size of the service line. In recent years there has been a recognition of this, and operators have tended to install larger services—particularly for commercial and industrial accounts. Changing consumer demand plus the high cost of replacing services have furthered this trend.

James G. Carns Jr. (*1*) of the American Water Works Service Company has said that the trend toward larger service lines is a very sensible approach for water works operators to adopt. He gives three reasons for such a procedure: [1] The number of large service installations is very small compared to the total number. [2] The cost to replace even a few inadequate services is out of all proportion to the cost of installing larger ones initially. [3] With adequate service size better water measurement can be obtained by selection of a suitable meter particularly when provision is made in the original installation for removing, repairing, and testing of meters. The wisdom of such installations is shown by the example of one building in which metered results showed two or three times the rate of demand shown in a similar one of the same size and with the same type of tenant. Office buildings, in particular, often show such a variation in demand.

Because of estimates and assumptions involved in determining sizes, there has been considerable divergence of opinion on this question. The divergence is increased by the lack of knowledge of the probable maximum demand of various types of establishments.

Even when data are available, good judgment requires that a margin of safety be provided for business expansion, technological changes, and similar contingencies. As far as meters are concerned, most water works operators realize that the best meter installation is the smallest one of proper design which can handle the range and frequency of change of flow rates experienced. This is particularly true because low-flow accuracy is relatively better in small-size displacement meters. The size depends largely upon the expendable pressure available and may be deter-

mined by learning the maximum demand and permissible pressure loss, and by referring to a capacity curve of the meter to be used.

Selection of the Meter

The selection of the proper water meter size, therefore, depends on a variety of factors and is not a simple problem. Many intangibles are involved in this phase of operation; local conditions of all kinds are factors and, in recent years, the problem has become further complicated by the increased use of water-consuming appliances including air-conditioning systems and flushometer fixtures.

Inadequacy of supply or insufficient residual pressure are generally blamed —without justification—on the size of the meter. According to some water works operators, such difficulties arise from the length and condition of the service line. Because of varying conditions to be found in each installation, it is virtually impossible to lay down a universal rule for determining the size of the service connection and meter. In addition to the length of the service connection, other factors such as pressure and the size and extent of the customer's plumbing enter the picture. Still others are the size of the shut-off valve and the number of fixtures. These elements have been used by various cities in developing elaborate methods of determining the size of house pipe and service connections.

The literature of water works operations shows that in normal times industrial installations take 30–50 per cent or more of all water sold by an average system. Even if the differential in rates is considered, the percentage of revenue coming into the system from these large meter installations is substantial. The procedure for determining the most suitable size of large meter is the same as for those of 1 ft or smaller. The maximum requirements of the customer and the rating of the manufacturer will give an indication of the proper size.

Usually, it is better to use a meter slightly smaller than the size of pipe supplying it. This practice is in line with the objective of securing the most accurate measure possible.

Use of the proper size meter is especially vital in apartment buildings and other multiple dwellings where only one meter is used. There, lack of a direct charge for water use tends to increase the per capita consumption.

It is not uncommon to encounter demands for large-size meters by architects and engineers in the design of large buildings. Although the demands seem somewhat exaggerated, they generally represent good design because they consider: [1] adequacy of distribution piping; [2] available pressure in main at service locations; [3] service line friction losses; [4] meter friction loss; [5] rate of water use; [6] house piping and fixture friction losses; and [7] height from main to highest water outlet. Such design is probably the result of the use of handbooks on hydraulics such as those published by the National Bureau of Standards and the Copper and Brass Research Association, plus experience, and consideration of growth factor. Often, one must accede to the architects and engineers by putting in a meter which it is felt, is too large for the particular service. Such situations should be subject to review at a later date and sufficient flexibility in meter installation should be provided to permit the size, type, and number of meters to be changed with a minimum of expense when changed service requirements require it.

Recording Registers

To determine whether a meter in service is of proper size, a recording register mechanism can be attached to the meter and data on the maximum and average demand rates obtained. There are several recording registers available, any of which would be a worthwhile investment for a water department. Their value can be illustrated by some examples from the author's experience.

A 3-in. compound meter serving a small hotel was not performing entirely satisfactorily and it was believed that a smaller size meter of another type would be more suitable. The hotel contained 150 rooms, 100 of which had complete baths. In addition, there were dining facilities and a hotel laundry. By use of a recording register, it was determined that the maximum rate of flow on a peak day was only 38 gpm. This, incidentally, occurred bebetween 7:00 and 8:00 AM. Reference to the manufacturer's loss-of-head charts showed that the loss of head at this rate of flow for a 3-in. compound meter was approximately 6 psi and the loss of head for a 2-in. disc meter was only 0.5 psi. As a result of this study, a 2-in. disc meter was installed and both the city and the hotel concerned received better service from the meter.

D. R. Taylor, general manager of the Baton Rouge Water Works Company in Louisiana recently stated that their four recording registers have been found indispensable. He reports that about a year ago a check was begun of consumption through 425 meters. Of the 400 already checked—which included meters from 1.5 in. up—80 have been found larger than necessary and been replaced with smaller meters. More were replaced with 1-in. meters than any other size. A few 1.5-in. meters, however, have even been replaced with ¾-in. size.

Conclusion

A knowledge of meters and their characteristics is essential for the water works operator who wants to select the best size meter for his specific installation. As is well known, a water system loses considerable revenue through improper recording at very low rates of flow because this condition is apt to prevail during a large part of the day.

Effectively determining the best size for both service and meter contributes greatly to securing payment for all the water being delivered. In the final analysis, it should be kept in mind that a larger meter can always be substituted if the consumer's needs require it, but it is much more expensive to replace a service.

Larger services will make it easier for the superintendent to fulfill his objective of supplying an adequate quantity of clean pure water to his customers, yet the smallest possible meter which will satisfy the demand will, in the end, result in the most accurate readings for maximum revenue.

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Charles Floyd Jr.

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As more and more water is required in modern living, it is becoming most important that careful consideration be given to the selection of a water meter that is the proper size to serve efficiently and without undue pressure loss. Not only must the size of the

water meter be considered, but also the size of service pipe, fittings, valves, and all other equipment that makes up the overall service.

Domestic Services

In the last few years there has been an increasing demand on domestic services which has resulted from the increase in use of water-using fixtures. This demand on the domestic service will continue to rise.

that reasonable rates for various water-using fixtures and appliances, are as follows:

Appliance	Rate—*gpm*
Laundry tub	5
Toilet tank	4
Kitchen sink	4
Bathtub	4
Shower	3
Lawn sprinkler	6
Washbasin	3
Total	29

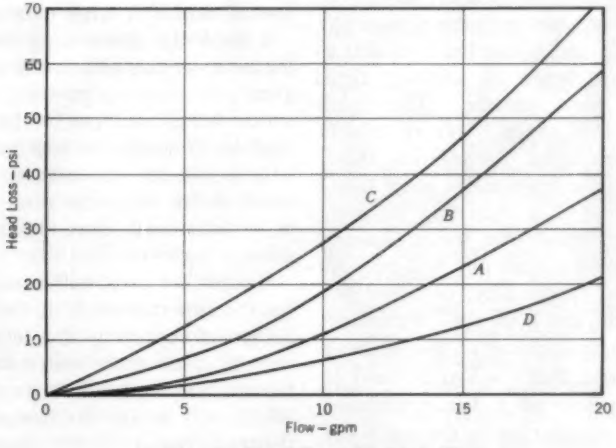

Fig. 1. Head Loss Through Meters

Curve A is for $\frac{5}{8}$-in. meter; B for a $\frac{3}{4}$-in.; and C, a 1-in.

Washing machines, air-conditioning units, dishwashers, garbage grinders, sprinklers, and many other devices once rare but now quite common in the home make the old idea of installing $\frac{5}{8}$-in. or $\frac{5}{8} \times \frac{3}{4}$-in. meters on all domestic services unsound.

Demand Rates

In order to give proper consideration to the size of the whole service assembly the demand rate on domestic services must be determined.

According to data given by Gerald E. Arnold (*1*), it has been determined

Although some authorities report that consumption rates for similar fixtures are considerably higher, it is felt that the figures given above are suitable for use in this discussion. Some dishwashers require 5 gpm with a residual pressure of 20 psi to operate satisfactorily; it is understood that new designs require considerably less than 5 gpm. Garbage grinders consume 2.5–4 gpm, depending on the make and model and, until recently, air-conditioning units of more than 2-ton capacity required water-cooling facilities of about 2 gpm per ton of capacity. Some

manufacturers have now designed units of 2-ton and more capacity that are air cooled, but possibly less efficient than water-cooled units. In view of this, the eventual effect of the introduction of air conditioning into homes is uncertain. For the purposes of this article, these figures can be considered:

Appliance	Rate—*gpm*
Automatic washer	5
Dishwasher	2
Garbage grinder	3
Air conditioner	6
Total	16

(Fig. 1). If loss of head through the meter were the only concern, there would be practically no problem, and it might be concluded that a $\frac{5}{8}$-in. meter would be satisfactory for nearly all domestic services. The pressure loss through the meter is, however, only one of the several factors to be considered, because there are losses in the corporation stop, service line, curb stop (or yoke), and other points.

If it is assumed that the average water service line from main to house is 60 ft, then with $\frac{3}{4}$-in. copper tubing

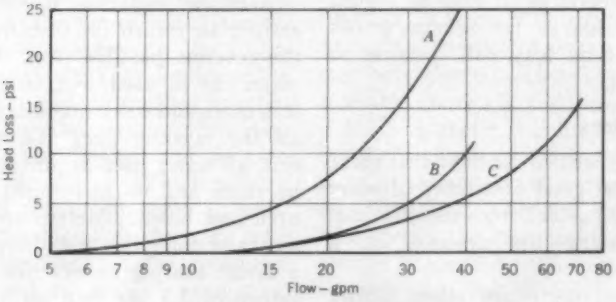

Fig. 2. Head Loss Through Full Service Lines

Curve A represents 60 ft of $\frac{3}{4}$-in. copper tubing; Curve B, a $\frac{3}{4}$-in. full service line with a $\frac{5}{8}$-in. meter; C, a full service line with meter horn and old meter; and D, a 1-in. service line with a $\frac{3}{4}$-in. meter.

Adding this 16 gpm to the previous 29 gpm total gives 45 gpm. It is a remote possibility that all units would be used at once, but it is reasonable to expect any of several combinations of the above units to be used at the same time. Such combinations would give approximately 15 gpm, and therefore, it can be concluded that domestic demand should be fixed at about 15 gpm for many of today's services.

Loss of Head

The loss of head through a $\frac{5}{8}$-in. meter at 15 gpm is only about 4.5 psi there will be a loss of about 23 psi in that line (Fig. 2). Loss of head through 60 ft of full service line—including a corporation stop, curb stop, and a new water meter—at 15 gpm is shown to be about 38 psi. Loss of head when a meter setter and an old meter are used in the 60 ft full service line is about 47 psi at 15 gpm; with a 60-ft 1-in. service line and a full $\frac{3}{4}$-in. water meter at 15 gpm it is only some 11 psi. There is generally more loss of head in an old unrepaired meter than in a new or repaired one. Head loss also varies according to type and brand.

The loss-of-head curves of Fig. 1 are for the split-case type of meter.*

Figure 1 shows loss of head curves for three different sizes of meters—⅝-in., ¾-in., and 1-in.—and the varying losses at the different flows.

It can be seen, then, that in selecting the proper size meter for a residence, it is almost essential to consider the overall service. Obviously, loss of head through the overall service is an entirely different matter from loss of head through the meter only.

Most of the data for the loss-of-head curves were obtained with new equipment and, therefore, it may be logical to assume that as the service grows older, the head loss will increase to some extent.

Other Factors

It may be well to consider that there is always the good possibility of more fixtures being added from time to time, thus increasing the demand on a service.

Of course, there are other factors to be considered in determining the meter size; factors such as adequacy of distribution lines, pressure at the main, house piping, fixture friction losses, and height from main to highest water outlet.

It is not possible to make precise recommendations on the proper size of a domestic service, but it does seem that for many present-day domestic services, it would be most practical to use a full 1-in. service line with a full ¾-in. meter or, in some cases, a full 1-in. meter. Specific circumstances would determine the exact size that would be practical.

When 1-in. service lines and fittings are used the cost goes up materially. Even in view of the higher first cost involved, it may be more economical to install a service of sufficient size rather than engage in replacement shortly after installation.

* Meter discussed is product of Badger Meter Manufacturing Co., Milwaukee, Wis.

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Accurate metering is very closely related to return on investment. All the revenue possible from the sale of water can be used, and, in these times it is more and more necessary to secure all the revenue due. To accomplish this, all water used by the consumer—no more and no less—must be measured and billed. Meters are the only means of doing this and, therefore, have a direct bearing on revenue. With a return of 3.3 per cent on investment, an inaccuracy of 2 per cent on the slow side could mean a loss of 6.3 per cent net profit.

Adequacy

Up to the present the industry has been fortunate in that, for most residential services, the ⅝-in. or ⅝ × ¾-in. meter on a ¾-in. service line has been adequate. This will be true for some types of services for quite some time to come, but there is a growing demand for larger services in many modern homes. Larger services do not necessarily mean larger meters. On residential services the size of the meter could and should be one size smaller than the service line. The head loss through a meter is comparatively small when compared with the

total head loss for the service. In fact, the length, size, and type of the service pipe is the main item of concern in head loss in any particular service. Generally, the smaller meter will be more accurate on smaller flows. Small flows on residential services can be of considerable value when it comes to profit and loss on residential services. The list price on a $\frac{5}{8} \times \frac{3}{4}$-in. meter is $46 and on a 1-in. meter, $96.50. The 1-in. meter represents an increase in investment of 110 per cent.

New-Service Routine

In the past, very little attention has been given to the size of water connections the customer receives. As a rule, a customer will apply for service at the business office and inform the clerk that he is building and wants a $\frac{3}{4}$-in. tap. An order is put through and the customer pays the charge. The next time he is heard from, "pressure is low," the bill is "too high," or there are taste and odor troubles. If the complaint concerns pressure, it will, as a rule, end as a combination of troubles. Further investigation will usually show that the customer has added more and more appliances and the coefficient on the service line and house piping has changed since its installation. Demand has changed from 5 gpm to 10–12 gpm and automatic equipment has been installed now which allows showering while washing dishes, washing clothes, or, perhaps, watering the lawn. If the original was $\frac{3}{4}$-in., then a 1-in. would in all probability correct the trouble. At the outset this would have cost $30.35 as compared with $21.87 for a $\frac{3}{4}$-in.—based on list prices. After carefully discussing with a customer the number and type of appliances he plans to use, he should be advised of the size of tap necessary. Advice on the size of pipe to use in the house plumbing can help give good service, save money, and make a satisfied customer.

Large Services

Perhaps the kind of service which is most consistently oversized or overmetered is the large ccommercial or industrial one. The tendency is to undersize the residential and oversize the commercial service. Recording registers can be used to check the demand of all larger installations in order to determine whether or not meter size can be reduced. There can be considerable savings in initial capital outlay if a 1.5-in. meter, for instance, can be installed instead of a 3-in. service. In most situations, such commercial meters have been called for by architects who specify the same size meter as the service line which they have computed for the particular installation. What water works operators need from the architect or his customer is data on the amount of water that will be required at any one time—that is, demand and the customers' honest appraisal of future needs. From such information the meter size can be intelligently prescribed and the service line size necessary to render long and adequate service to any customer computed. Others have found that as much as 25 per cent more will be registered when the larger meter (3-in. and larger) is replaced with a smaller one, commensurate with the demand. Special attention should be given to apartment dwellings, hotels, and schools.

Meeting Greater Demands With Larger Pipe

A. M. Friend

A paper presented on Sep. 18, 1957, at the Ohio Section Meeting, Cincinnati, Ohio, by A. M. Friend, Cons. Engr., R. L. Woolpert Co., Dayton, Ohio.

SINCE the end of World War II, the state of Ohio as well as the rest of the country has participated in a construction program unparalleled in its entire history. This program, including all phases of building, is a result of many factors. These factors include normal increases of population, additions and expansion of industry, and the trend of movement of people from the towns and cities to the suburban areas, where there is more living space for larger and more prosperous families (Fig. 1). It is predicted that this latter trend will continue at an accelereated pace in future years.

Estimates for the changes in population in the United States indicate an increase of 5 per cent in the next 5 years, a 17 per cent increase in the next 10 years and a 36 per cent increase, or one-third more people, in the next 20 years. These increases will require similar expansion of all related facilities. None of these increases can be supported without a safe and dependable water supply; therefore, the owner of a water utility, be it public or private, must plan not only for the forseeable problems, but also for future requirements.

The water utility consists of the source of supply and the treatment, pumping, storage, and transmission and distribution systems. The transmission and distribution system comprises the largest single item and the most costly one of the utility. The network of pipes is underground and out of sight, and this may be the reason that, in many cases, the same care is not exercised in an understanding of all of the items to be considered in the selection of the sizes of pipes that actually carry the water to the consumer as in the selection of pumps, size of treatment units, and amounts of storages. These considerations are, in effect, the most important, as it is more difficult and expensive to increase the capacity of the transmission and distribution systems after they are once installed than the other integral parts of the utility.

The capacity of the transmission and distribution system, or the size of the important main lines in the system, is based upon hydraulic and economic considerations. The system should contain enough capacity to provide adequate service during periods of peak demand to all consumers; also, the cost and operation of the system should be within the available means of financing. A careful review of both of these considerations in respect to their close relationship is required before final selection of size of the trunk water main is made, to determine the most effective and economical pipe size for the system.

Hydraulic Considerations

The selection of pipe sizes for the distribution network requires careful calculation, coupled with experience and practical knowledge. The pipe sizes are usually determined after assuming definite limits of pressure drop within the system. This variable working pressure range can be determined from the topography of the system, location and elevation of source of supply or booster station, and the location and elevation of storage within the system.

also maximum pressures to which a system can be subjected and not cause damage to services and internal plumbing. Pressures in excess of 100 psi are too high for domestic service.

The pressure drop during the operating cycle is confined to a relatively small range with limits on the maximum high at the lower end of the system and minimum low in the high portions of the system.

The difficulty experienced in placing additional load on existing systems has been due to insufficient capacity in the

Fig. 1. Housing Development Near Dayton, Ohio

This 1,800-home development, outlined in white, is an example of the construction that places additional heavy demands on water distribution systems, as more people seek to live in suburban areas.

The allowable pressure drop within the system is limited to that required to provide adequate service to the most remote service or to the consumer at the highest elevation on the day of maximum demand. A 40-psi water pressure can be considered as the minimum for adequate service for the average resident under the maximum conditions. The allowable pressure drop may also be governed by residual pressures at outlying fire hydrants during periods of fire flow. There are

original systems, which results in high friction losses. In many cases more capacity could have been installed initially at little additional cost.

Two basic laws concerning flow in pipes are: [1] at a constant head the capacity of the pipe is proportional to the diameter raised to the 2.63 power; and [2] at constant diameter the friction head is proportional to the discharge raised to the 1.85 power.

The first law is illustrated by comparing the increased capacity with ex-

pected discharge through a pipe one size larger than specified at the same friction head loss and same roughness coefficient. For example, if 600 gpm is carried in a 6-in. pipe, then 1,270 gpm will be carried in an 8-in. pipe under the same conditions of head loss. Likewise, a flow of 800 gpm in an 8-in. pipe under the same conditions of head loss will produce a 1,440-gpm flow in a 10-in. pipe; under the same conditions, 1,200 gpm in a 10-in. pipe will increase to 1,940 gpm in a 12-in. pipe, 1,600 gpm in 12-in. to 3,390 gpm in 16 in., 2,000 gpm in 16-in. to 3,600 gpm in 20-in., and 2,400 gpm in 20-in. to 3,880 gpm in 24-in. pipe. Therefore, by increasing the size by one pipe size, the flow capacity increases 160–210 per cent.

The second law states that in any given pipe, if the discharge is doubled, the friction loss is 360 per cent that of the first condition. For example, if the loss in a 6-in. pipe with a coefficient of 100 is 6.2 ft per 1,000 ft for a discharge of 200 gpm, it will be 22.1 ft per 1,000 ft for a discharge of 400 gpm.

If a distribution system is near the limits of its allowable range of pressure drop during periods of peak demands, very few if any additional customers can be added without revisions or additions to that system.

Economic Considerations

There are many factors affecting the cost of the construction of a water distribution system that are outside the actual cost of installing the pipe. These include the type and amount of surfaced street, sidewalk, curb, and lawns to be removed and replaced; the type of backfill material, such as pea gravel, selected pit-run gravel, or sand; the number of fire hydrants; and the number and type of interconnections between the existing system and the new lines if the project consists of new trunk mains to support an older system. The actual costs of these items for pipe sizes 12 in. and less are very near the same, and the cost increase for these items on larger size mains when comparing two pipes, one size apart, is very small.

The installation of a new trunk water main through an existing system requires many interconnections with the existing system. If there are no convenient tees and crosses in the existing system to receive these connections, as is the usual situation, additions to the existing system are then required. The costs of these additions to the existing system are independent of the size of the new main from which they will be supplied.

Analyses of many bids on water distribution projects in the Dayton, Ohio, area indicate that the costs not pertaining directly to the size of water main, fittings, and valves approximate 17 per cent of the total construction costs. This figure is for average conditions with bituminous surface–treated streets and small sections of curb and gutter. This item will increase to more than 20 per cent of the total cost with better street surfaces and base courses.

The manufacturers of water main materials have kept pace with progress, and many notable changes have been accepted within the last decade. For example, they have partially offset the rising spiral of labor costs by the introduction of joints which eliminate the more expensive process of pouring and calking. The actual material price with a mechanical joint is more than the bell-and-spigot type of pipe, but the cost of assembling the pipe reduces

the labor cost so that the overall cost of the complete installation is less.

New rubber seal joints have recently been introduced, and they have already proved effective. The material cost of pipe with this type of joint is some 5–7 per cent less than the mechanical joint, and less labor is required in making up the pipe. Recent bids utilizing this material have already reflected a reduction in the installed pipe cost.

The adoption of the AWWA method of design for cast-iron water mains using more realistic criteria has resulted in the savings in the amount of metal in pipe in sizes of 8–20 in. This saving results in a reduction in material and handling cost.

These material improvements comprise much of the reason that the cost of installation of water mains has increased only 8–10 per cent in most parts of Ohio since 1951, whereas the entire construction cost index for the country, as compiled by *Engineering News-Record* at the end of 1956, shows an increase of 27 per cent for the same period.

The bids on 22 water projects, all received within the last year, were analyzed to determine the present cost to install water mains in the Dayton–Montgomery County area. These projects are both in traveled streets and upon undeveloped land. They include the cost of pipe and fittings and excavation and backfill, or excavation and removal of excavated material. In the latter situation the backfill would be paid under an item of gravel trench backfill. The prices do not include valves or hydrants. The averages of the low bids are as follows: $3 per foot for 6-in. pipe, $4 for 8-in., $5 for 10-in., $6.50 for 12-in., $9 for 16-in., $12 for 20-in., and $16 per foot for 24-in. pipe.

In the above figures, the 6–12-in. sizes are ASA Thickness Class 22 cast iron, and the 16–24-in. sizes are of prestressed concrete. These figures are quoted so that the differential in costs between the various sizes of pipe may be shown.

For a particular example, a water distribution system was recently completed in the Northridge Sewer District for the Montgomery County Commissioners at a construction cost of $936,380.88. This is an average project, which covers most phases of distribution construction. The project consisted of 0.5 mile of 24-in., some 16-in., 5.6 miles of 12-in., 4.1 miles of 10-in., 2.0 miles of 8-in., and 22 miles of 6-in. pipe. The actual cost of furnishing and installing the cast-iron water mains, including valves and fittings but not hydrants, was the following: $3.31 per foot for 6-in. pipe, $4.07 for 8-in., $5.13 for 10-in., and $6.80 for 12-in. pipe.

The cost on this particular project pertaining only to the water mains and valves was $778,647, or 83 per cent of the total. The remaining $157,700 of the construction cost was for other items of work which would be required in any case, regardless of whether several of the water mains were changed one size on the design of the system.

It is interesting to note from the above figures the results of changing the design of this system. For instance, if all the 12-in. mains had been reduced to 10-in. mains, the construction cost of the project would have been reduced by $49,710. This appears to be a large figure, but on a project of this size the reduction in construction cost would be but 5.3 per cent. Under these conditions, however, the capacity of the main loop within the system would have been

reduced by one-third, with a resulting increase in power cost for the pumping stations. A more reasonable analysis would be the effect of changing only sections of the 12-in. main to 10-in. pipe. This cost would be $8,810 per mile, or less than 1 per cent of the construction cost of the project. Similar comparison can be made on the other sizes of water main, showing that a decrease in size of mains in some sections of the project will decrease the total cost of the project by only a small percentage.

Future Planning

The water utility planner must investigate different designs for a water distribution project, whether the project is a complete system in itself or a simple one-line extension that will be integrated into the system at a later date. It is possible to provide additional capacity to a system by installing larger mains on routes which are not presently served with water and still intersect the existing grid of water mains at enough points to support the interior system. It is poor economy, on the other hand, to parallel existing water lines in surfaced streets to provide this additional capacity, as some 15–20 per cent of the cost is duplicated in the initial installation. This amount of money could have financed some increased pipe sizes on the first project.

Some duplication of facilities is required, due to important changes in the land use of the area. Others are required when physical relocations are necessary as the result of highway construction. The highway program in Ohio is affecting many water systems, and the construction cost of this type of work is large, particularly if the new highway is depressed. Much of the cost is due to special construction and the requirement that the old line must remain in service while the new line is installed. In this type of work, the actual pipe cost can be increased without adding too much proportionally to the total cost, as the other costs are so high. It is of importance to have mains of sufficient capacity in these locations, as any future revisions would be very costly.

The means that are available to finance all of the extensions or improvements to a water system that may be desirable are sometimes limited. It is the duty of the utility planner to see that the available funds are expended in such manner that the system receives the most possible benefit. By increasing some of the water lines one pipe size, capacity can be increased by a percentage far in excess of the costs of such changes.

A substantial distribution system is a large part of the water utility that is necessary to provide an adequate supply of water and fire protection at all times to the consumers. An adequate system is not only desirable but necessary, so that the owners and operators of water utilities can keep pace with the country's total expansion.

Applications and Standards for Asbestos-Cement Pipe

Philip H. Perkins

A contribution to the Journal by Philip H. Perkins, Civ. Eng. Consultant, Nicosia, Cyprus.

ASBESTOS-cement pressure pipe has been used for water supply work for many years in various parts of the world. If the class of pipe is properly selected in relation to the operating conditions, these pipes give very satisfactory service.

Corrosion

The usual corrosion problems—both internal and external—which arise with the use of steel and cast-iron pipe do not occur with asbestos-cement pipe. It is possible, however, that, if there is a very high sulfate concentration in the ground in which the asbestos-cement pipe is laid, sulfate attack may occur. External bituminous coating is helpful in such situations. In Kuwait * on the Persian Gulf, where corrosion of steel and cast-iron pipe created a very serious problem, asbestos-cement pipe was used, with the expectation that it would not be affected by sulfate attack. Because of the interior smoothness and resistance to corrosion of the pipe, it maintains its initial hydraulic characteristics over a long period.

Types of Joint

There are two principal types of joint used for general water supply work with asbestos-cement pipe—a friction joint.† and the Gibault-type joint. The former consists of an asbestos-cement sleeve with rubber rings between the sleeve and the pipe. It is best assembled by means of a special joint-mounting tool. For large-diameter pipe, this tool must be used if a satisfactory joint is to be made. The latter, which does not require as careful assembly as the friction type, consists of a steel middle ring with rubber rings between it and steel followers. More easily assembled than the friction type, it is more expensive and, unless precautions are taken, it is subject to corrosion of its cast-iron portions.

Other joints are a threaded type for 2–6-in. diameter pipe,‡ a monolithic one for Class *B* § pipe up to 6 in. in diameter, a flexible lead joint ‖ for pipe

* Operating pressures, 30–150 ft of head.

† The Simplex joint, a product of the Johns-Manville Corp., New York, N.Y. It consists of an asbestos-cement sleeve which fits over the end of the pipe and two rubber rings which are compressed between pipe and sleeve.

‡ Product of Turners Asbestos Cement Co., Ltd., Manchester, England.

§ Classification shown, used in British standards, corresponds most nearly to AWWA Class 200. (Exact pressures: Class *B*, 172 psi; Class 200, 198 psi.)

‖ Both the monolithic and the flexible lead joints are manufactured by Italit Inc., New York, N.Y.

which conveys liquids and gases which may have a deleterious effect on rubber jointing rings, and two other ring type joints.*

Specials (such as tees and bends) are usually of cast iron and connected to asbestos-cement pipe by means of Gibault joints. Sluice valves inserted in the pipelines are sometimes connected by the same means, but the best practice is to use flanged valves and special flanged adaptors to connect the valve to the pipeline. End caps are of cast iron and should be securely fastened in position with concrete, and all bends and tees should also be securely anchored by means of concrete blocks.

Precautions in Laying

When asbestos-cement pipe is laid in rocky ground, or ground containing a large percentage of stones, it is essential that a cushion of sieved earth or sand be laid on the bottom of the trench to a minimum depth of 4 in. The first 6 in. of fill around and over the pipe should also be of this material to avoid having large stones come in contact with the pipe.

Operational Tests

The hydraulic pressure tests given in the various standards mentioned below, all refer to the tests carried out in the factory. It is obviously essential, that the pipe, when laid, be tested to such a pressure as to provide an adequate margin of safety under operating conditions. It is left to the design engineer to decide the test pressure to be applied to the completed pipeline. It is usual practice for this operational test pressure to be 50 per cent above the working pressure specified in the appropriate standard. For example, if the engineer is working according to the specifications of British Standard 486, a Class *B* pipeline would be tested to 300 ft of head (130 psi)—that is, 200 ft of head working pressure plus 50 per cent.

When carrying out an operational test, the pipe should be filled with water for at least 24 hr before the pressure test is started; the pressure should then be slowly and steadily applied until the specified test pressure is reached. The period of test under maximum pressure should not exceed 30 min. Operational pressure tests are usually carried out on lengths not exceeding 500 m (1,600 ft).

A further test which should be applied to long pipelines is a leakage test, usually applied to the completed pipeline. It is customary to specify the maximum loss of water which is permitted under the working pressure per inch of diameter per mile per test period of 24 hr.

A typical specification for carrying out operational pressure and leakage tests on new pipelines is shown in the appendix.

Liability for Damage

Probably the chief disadvantage in the use of asbestos-cement pipe is its liability to damage when subjected to rough handling. Where the pipe must be shipped to its destination and where handling conditions are likely to be severe, special precautions must be

* The Ring-Tite coupling, a product of the Johns-Manville Corp., consists of an asbestos-cement sleeve and two rubber sealing rings which, compressed into grooves machined into the sleeve, form a flexible joint. The Fluid-Tite coupling, a product of the Keasbey & Mattison Co., Ambler, Pa., consists of an asbestos-cement sleeve and two rubber rings which are self-energizing under pressure and which fit internal grooves in the sleeve.

taken to protect the pipe—particularly the ends. If the pipelines concerned are important ones, it is advisable that pipe should be specified at least one grade higher than that which would be required by hydraulic considerations alone.

In Kuwait long lengths of 18-, 16-, and 12-in. diameter Class *B* asbestos-cement pipe made to continental standard thicknesses were laid between 1952 and 1955. Considerable difficulties were experienced in testing these pipelines to the required specification—50 per cent above the working pressure, or 130 psi. The matter was very carefully investigated by the consultants, the state water department, and the manufacturers of the pipe and there seemed to be very little doubt that the pipe had suffered severe damage from very rough handling received when it was landed at Kuwait. This damage was often quite undetectable until the pipe had been laid and the line tested. There were occasions when pipe previously tested to 60 psi failed on a subsequent test of 50 psi. Subsequent orders for pipe specified a wall thickness corresponding to that laid down in South African Standard 286 (1951). Table 1 shows comparative wall thicknesses for which pipe conforms to various standards.

Standards

In the United Kingdom, British Standard 486 (1933), dealing with asbestos-cement pressure pipe, provided for four classes of pipe—namely, Classes *A*, *B*, *C*, and *D*. These classes correspond to the following specified factory test pressures: Class *A*, 200 ft of head; Class *B*, 400 ft of head; Class *C*, 600 ft of head; and Class *D*, 800 ft of head. It was further specified that the working pressure should not exceed half the respective test pressure.

This standard has just been revised and is now British Standard 486 (1956). The general requirements in the latest revision follow very closely the original specification. The important changes are:

1. In Table 1, the pipe thicknesses have been reduced.
2. The tables of classification of dimensions and tolerances have been duplicated, with the second series providing metric equivalents.
3. The test requirements have been increased and an additional appendix sets out precautions recommended for the fixing of ferrules.

In the United States there is Federal Specification SS-P-351a, dated Oct. 7, 1953, and the tentative standard issued by the AWWA (*1*) dealing with asbestos-cement pipe intended to be used in water distribution systems.

The federal specification divides the pipe into three classes: 100, 150, and 200, corresponding to working pressures of 100 psi (230 ft of head), 150 psi (350 ft of head), and 200 psi (460 ft of head). The factory test pressures are: Class 100, 800 ft of head; Class 150, 1,200 ft of head; and Class 200, 1,600 ft of head. The specification also lays down requirements controlling the permissible variations in thickness, length, and diameter. There is no requirement regarding minimum thickness of pipe walls and this is apparently left to the manufacturers to decide. The AWWA Standard follows the same general lines as the federal specification, and the classification and test pressures are identical.

In South Africa, Standard 286 (1951) applies to asbestos-cement pipe and is more closely allied to the British standard than the American and

TABLE 1

Wall Thicknesses of Asbestos-Cement Pipe Specified by British, South African, and Continental Standards *

Pipe Diameter in.	South African Standard 286 (1951)			British Standard 486 (1933)			British Standard 486 (1956)			Continental Standard†		
	Class B	Class C	Class D	Class B	Class C	Class D	Class B	Class C	Class D	Class B	Class C	Class D
4	0.39	0.44	0.51	0.47	0.61	0.66	0.42	0.50	0.50	9 (0.36)	10 (0.40)	12 (0.48)
6	0.43	0.55	0.69	0.61	0.78	0.90	0.45	0.61	0.70	11 (0.44)	13 (0.52)	16 (0.64)
8	0.55	0.71	0.88	0.72	0.96	—	0.57	0.72	0.90	13 (0.52)	16 (0.64)	20 (0.80)
9	0.57	0.79	0.98	0.79	1.05	—	0.60	0.79	0.98	—	—	—
10	0.60	0.85	1.08	0.84	1.16	—	0.64	0.84	1.08	15 (0.60)	19 (0.76)	25 (1.00)
12	0.73	1.02	12.7	1.00	—	—	0.68	1.00	1.30	17 (0.68)	23 (0.92)	30 (1.18)
14	—	—	—	1.15	—	—	—	—	—	19 (0.76)	27 (1.06)	35 (1.38)
15	0.89	1.24	1.55	1.23	—	—	0.84	1.24	—	—	—	—
16	—	—	—	Not available	—	—	—	—	—	23 (0.92)	32 (1.28)	42 (1.68)
18	1.06	1.46	—	1.47	—	—	0.95	1.47	—	—	—	—

* All thicknesses in inches unless otherwise specified.
† Thicknesses in millimeters. Figures in parentheses give equivalent in inches.

continental standards. The minimum wall thickness of the pipes is specified. In general, these are intermediate between British Standard 486 (1933) and the continental standard referred to below.

On the continent, asbestos-cement pipe is generally manufactured to a standard prepared by the International Standard Organization. This standard divides the pipe into two classifications —*A* and *B*. Classification *A*, which is usually adopted for water supply work, is subdivided into four classes corresponding to those in British Standard 486—that is, Classes *A*, *B*, *C*, and *D*. The continental standard also states that the working pressures shall not be greater than half the hydrostatic factory test pressure applicable to any particular class of pipe. The thickness of pipe walls is not specified, reference being made to the thicknesses established by the manufacturers. Provisions are included, however, for controlling variations in wall thickness, diameter, and length.

Pipe Wall Thickness

As stated above, two standards—British Standard 486 and South African Standard 286 (1951)—give the minimum thickness of the walls of pipe of various diameters and classes. This controlling clause is not included in the American standard nor in the continental standard. It should be noted, however, that the factory test pressures and the working pressures referred to in the United States standards are considerably higher than those required by the British, South African, and continental standards.

Probably the most controversial point regarding the use of asbestos-cement pipe is that of the thickness of the pipe wall. There is considerable divergence of opinion among engineers as to whether a standard should specify minimum thickness or not, and whether the extra thickness required by the British standard over that provided by continental manufacturers is justified or not. In the author's opinion, the extra thickness is justified only where handling conditions are likely to be particularly severe.

Reference

1. Tentative AWWA Standard for Asbestos-Cement Water Pipe—AWWA C400. Am. Wtr. Wks. Assn., New York (1953).

APPENDIX

Requirements for Testing Installed Asbestos-Cement Pipe (Author's Requirements)

Completion of Main-Laying Work

The section of main to be tested shall be completed before the tests are carried out—that is, all sluice valves, air valves, washout valves, and branch connections must be installed. Thrust blocks and any holding down anchorages must be completed. Air cocks (¾-in. diameter) should be placed on each side of any main sluice valve (10-in. size or larger) and used to release air before and after testing.

Testing of Mains

Mains shall be carefully tested and test pressures applied in the manner and to the limits described below.

a. Hydraulic Pressure Test

The main shall be filled with water and slight pressure applied. This must then be left for a minimum of 24 hr. During hot weather the period should be not less than 48 hr. After this any loss of water

will be made good and the actual pressure test started. The pressure shall be steadily applied until the specified operational test pressure is reached. The test pump must then be removed. After 30 min, the pressure in the main shall be again measured and the drop in pressure during this 30-min period must not exceed 10 per cent of the original test pressure. The operational test pressure to be applied shall be as follows for the various classes of pipe stated:

Pipe Class	Working Pressure (Half the Factory Test Pressure)		Operational Test Pressure (Working Pressure Plus 50 per cent)	
	Feet of Head	*psi*	*Feet of Head*	*psi*
B	200	87	300	130
C	300	130	450	195
D	400	175	600	260

b. Leakage Test

This test will be applied in addition to the hydraulic pressure test described above for pipelines which exceed 1,000 m (3,300 ft) in length. It shall be applied to the whole of a completed pipeline and shall be carried out as follows: The main shall be filled with water and slight pressure applied. This should then be left for a minimum of 48 hr. After this, any loss of water will be made good and pressure steadily applied until the working pressure as specified above is obtained. The test pump shall then be removed and the main left for 24 hr. After this period the quantity of water which has to be pumped into the main in order to regain the original pressure shall be measured, and this quantity shall not exceed 40 gal/in. of internal diameter per mile.

Inspection and Test Report

Careful inspection shall be made during any test to observe any movement, seepage, or damage. A comprehensive test report shall be prepared for each test.

Chlorination of Mains

As directed by the engineer, the mains shall be chlorinated as follows: All test water shall be run off and the main thoroughly washed out. Highly chlorinated water shall then be pumped in and kept in the main for 24 hr. The free-chlorine content of the water at the end of this period must be not less than 1 ppm. After satisfactory completion of the chlorination, the mains shall be considered fit for final commission.

Improvements in Lime Handling, Feeding, and Slaking at Lansing, Mich.

John F. Dye

A paper presented on Sep. 26, 1957, at the Michigan Section Meeting, Detroit, Mich., by John F. Dye, Plant Engr., Water Conditioning Plant, Board of Water & Light, Lansing, Mich.

LIME has been made and used by mankind for a good many hundreds of years as a building material and as a basic chemical for the manufacture of many other materials. Only in the last 40–50 years, however, has it been found necessary to produce large amounts of lime of constant high purity and to develop means of handling it in such a way that this purity is protected and preserved. The handling and conveying of chemicals and other materials has developed until it is a very important branch of engineering.

Lime for softening water can be used in the form of calcium oxide, or quicklime, or in the form of calcium hydroxide, or hydrated lime. The smaller industrial types of lime-soda softeners, as a rule, use hydrated lime, but the greater proportion of municipal lime-soda softeners use quicklime, and it is used in such quantities that it is usually purchased and shipped in bulk. Twenty years ago this lime was shipped in boxcars equipped with grain boards at the doors to prevent loss. Today a shipment of quicklime in a boxcar is a rarity, covered cement hoppers of 45–50-tons capacity being almost universally used for this purpose, because of the protection they provide for the lime and the ease with which they can be unloaded. In a few places, such as Ann Arbor, Mich., where there is no railroad siding at the softening plant, container cars have been used, the containers being trucked to the plant for unloading.

Conveying the lime from the car or container is now no problem. In some of the older plants the lime is discharged from the hopper car to a bucket conveyor system which carries it to the bins. In most modern plants built in the last 25 years, however, the pneumatic conveyor is preferred. There the lime is discharged into an air stream which carries it to a large, coned-bottom separating tank. The air is drawn by a blower through cloth filter bags in the separating tank to remove the dust and then discharged to the atmosphere. The lime is fed from the bottom of the tank by a rotary valve to a screw conveyor which, in turn, carries it to the proper bin. In the Lansing, Mich., plant, where conveyor equipment was installed in 1939, there are six lime bins, six soda ash bins, and two alum bins. Motor-operated valves or gates, with controls located on the operating floor of the plant and also in the room on the same level as the top of the bins, allow the operator to select the bin into which the chemical is to be discharged.

Handling Equipment

In many plants it would be advantageous to be able to transfer lime from one bin to another. At Lansing,

three of the six lime bins are located above the three feeders in the original plant. The two flocculator and clarifying basins for which these feeders were installed are now used as secondary basins where no lime is required. When the new primary basins were placed in service in 1949, the bins above the older feeders were full of lime. Shortly after this, to make this lime available, plans were made to install a 6-in. manifold, extending from the suction line to the separating tank over the bins, with a stub lateral for each of the three old bins. Flexible metal hose was to be attached to the lateral above one of the bins so that the lime could be pulled up out of the bin to the separating tank and conveyed out to any one of the bins above the new feeders.

Work was already started on this when a railroad strike occurred, and no lime could be shipped. One of the new lime bins was empty and the other two were dangerously low when the installation of the transfer system was completed and then tried out. The system worked, and it was gratifying suddenly to have another 450 tons of lime on hand.

Bins for storage should be located directly above the lime feeders. Handling of pebble quicklime develops a fairly large amount of fines, which cause the lime to hang up in the corners at the junction of the vertical sides and the slanting bottoms of the hoppers. The sides of the hopper bottoms should be at an angle of not less than 60 deg with the horizontal. If they are much flatter, the result is that an appreciable amount of the lime shown on the plant inventory is not available for use without a lot of disagreeable labor.

Advantages of Hydrated Lime

As mentioned above, lime can be used in water plants in the form of quicklime or in the form of hydrated lime. The latter form is sometimes used for pH adjustment when an increase in hardness is not objectionable, and, as a rule, industrial lime-soda ash softeners use hydrated lime. Since the amounts used are smaller than for municipal water softening, the fact that it stores better than quicklime is an advantage, allowing the lime to be purchased in carload lots, even though this amount may be sufficient for 6 months to 1 year. Although hydrated lime is safer to handle than quicklime because its affinity for water has been satisfied, nevertheless, it is a highly alkaline material, and dust masks must always be worn by operators when filling hoppers or making up a new batch of slurry.

Another advantage in using hydrated lime is that it does not require slaking. It may be fed from a dry feeder into a mixing tank or it can be made up as a slurry, which is pumped at a rate proportional to the flow of the water being treated. When used for pH adjustment, automatic pH control is available, eliminating manual changing of the feed rate. Altogether, hydrated lime has its place in the water treatment industry, and the feeding of this material need no longer be the hit-or-miss proposition described by Lordley (*1*) in 1945.

Most municipal lime-soda ash softeners use quicklime because of its lower cost. Usually it is shipped in bulk and stored in bins above the feeders, so that it can be fed directly to the weighing device. As a rule, rotary kiln pebble lime is preferred over shaft

kiln lime, because of its uniformity of composition and size and its quick slaking properties. The desirable limits for these properties and methods of testing are all given in the "AWWA Standard for Quicklime and Hydrated Lime" (*2*).

Feeding Equipment

The modern lime feeder bears little resemblance to the first rotating disc-type feeder that appeared on the scene with the advent of rotary kiln lime in the 1920's. In the Lansing Water Conditioning Plant there are six gravimetric lime feeders and slakers. Three of these were installed in 1939 and three in 1949, when the plant was expanded. Each of the feeders has a maximum capacity of 3,000 lb/hr and a minimum capacity of 400 lb/hr of quicklime. Each of the bins above the lime feeders has a capacity of about 150 tons of pebble quicklime.

A vibrating mechanism shakes the lime down onto a moving feeder belt that is balanced on a scale and calibrated to feed 20 lb for each foot of belt travel. The lime spills from the discharge end of the belt into the first compartment of the slaker. The intensity of the vibration that shakes the lime onto the belt is modified by the weight of the lime on the belt, so that the belt and its load of lime is balanced on the scales at all times. If, for any reason, the feed should continue to be too heavy or too light, an alarm sounds to indicate trouble. According to the specifications the alarm will sound whenever the feed rate changes ± 1 per cent.

Each feeder has a speed changer for manual adjustment of the speed of belt movement and, thus, the rate of lime feed. In addition, automatic control is provided, which adjusts the feed for changes in the raw-water influent rate. The control for this adjustment is taken from the raw-water venturi meters. The operator adjusts the speed changer for the required amount of lime feed for a full rate of 22 mgd into the primary basin. Then, with the controls set on automatic, the proportion of time on line that the belt moves is controlled by the raw-water influent rate through the venturi meter. Pneumatic signals from the venturi meter are transmitted to an automatic meter, which, by an ingenious cam arrangement, varies the electrical contact time for power to the solenoid-operated belt shifter that starts and stops the speed changer and feeder belt. This automatic control is the result of considerable experimental work at the Lansing plant and in its present form has proved very reliable. The only changes that the operator has to make in the rate of feed are those dictated by the results of laboratory tests on samples from the rapid-mix, flocculator basin, and clarifying basin effluents and by experience with water from the various well fields.

In the speed changers, as originally installed, small wheels ride on rotating cone-shaped cams. The position of these wheels can be changed lengthwise on the cams, thus varying the amount of motion transmitted through the clutch to the feeder belt. These wheels had a very narrow bearing surface and wore grooves in the cams. As a result, it was very difficult to make small changes in the speed of the feeder belt. For several years it was necessary to remove the cams periodically and have a welder fill the grooves. The cams were then dressed and smoothed and were as good as new

for a while. After this process had been repeated several times on each set of cams, the mechanic who made these repairs suggested that the wheels be replaced by valve lifters from a car motor. The flat surface of these valve lifters would have a greater bearing surface, and grooving of the cams would be eliminated. His suggestion was acted upon in one speed changer and it worked perfectly. The other speed changers were also altered in this way, and from that time no further maintenance on the cams was necessary. The manufacturer of the feeders immediately adopted the idea, and all of the speed changers for the eight feeders installed in the plant extension now have these valve lifters instead of wheels. In the 8 years since their installation, very little maintenance has been necessary on these changers, except the periodical changing of the oil in the oil bath in which the cams rotate. Figure 1 illustrates both the old and new methods.

Although the lime fed onto the moving belt is controlled primarily by the vibrator, as described above, the gate through which the lime moves onto the belt has to be adjusted for differences in the size of the lime. As originally furnished, this gate was very difficult for the operators to adjust. By installing racks on the gate and gears on the shaft extending across the gate and by placing a crank on the end of the shaft, it is now possible to adjust the position of the gate very easily (Fig. 2).

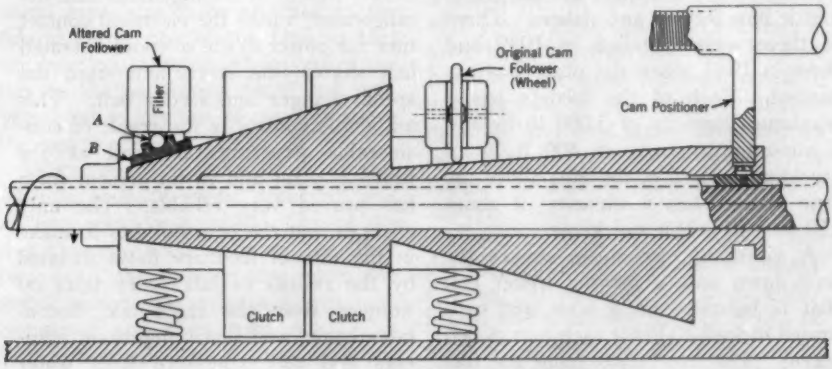

Fig. 1. Portion of Speed Changer

The drawing shows both the original cam follower, which wore grooves in the cam, and the altered model, designed to prevent the condition. Components include: A, *the ball bearing welded to the cam follower arm; and* B, *the valve lifter, with a greater bearing surface than the wheel.*

In the original plant, with the tendency of light soda ash to flush and run through the feeder like water, experience had led to the use of dense soda ash. When the extension to the plant was added, the new soda ash feeders were provided with antiflush devices. Dense soda ash continued to be used, however, and as a result, these antiflush devices saw very little use. On the other hand, cars of very fine lime were occasionally received. As

this fine lime feeds out of the bin, there is a tendency for a hole of 3–4-ft diameter to form, extending vertically from the surface of the lime down to the bottom of the bin. The fine lime forming the walls of this hole may suddenly cave in, with the result that it flushes in the same way that the light soda ash did. When this happens, the feeder belt compartment and the first compartment of the slaker become filled with quicklime, which, because of the steam from the slaker, becomes very hot and also forces open the doors of the belt compartment, spilling onto the floor around the feeder almost ankle deep. The glass in the doors is usually cracked, and the wood strips holding the glass are often charred. After several such experiences with the flushing of fine lime, the antiflush devices were removed from the soda ash feeders and installed in the lime feeders, where they have prevented several of these difficulties.

Slaking Equipment

The lime slakers at the Lansing plant are rectangular, round-bottomed tanks, divided into four compartments to prevent short circuiting. A propeller-type agitator in each compartment keeps the slurry well mixed during slaking. The slakers are large enough to provide a retention time of 30 min at the maximum feed rate. The last compartment is equipped with a heat exchanger.

The slakers at Lansing are among the first for which automatic control of the slaking temperature was specified. The specifications also called for heat exchangers so that the hot slurry leaving the slakers would warm the water to be used for slaking, thus retaining the heat in the slakers. By this method the slaking temperature is maintained at160°F–170°F, usually very close to the latter temperature. With pebble lime, if the temperature is allowed to rise above 170°F in the first compartment, the slurry becomes too thick for good agitation and the lime then piles up under the opening from the feeder compartment. It may even partially fill this compartment. A limited amount of water mixes with the lime, slaking it, but the heat is retained in the pile, with results about the same as described above when fine lime flushes into the feeder. If this occurs shortly after the operator has made his hourly readings, the alarm sounds when the lime has piled up under the end of the belt. This indicates that not enough lime has been fed. If the operator is lucky, he can stop the feed of lime and then thin out the mixture in the first compartment by opening an emergency water valve. Usually an occurrence of this kind results from faulty agitation of the slurry, caused by worn agitator impellers. It is, therefore, necessary to keep a supply of these impellers on hand and to replace the worn impellers whenever the agitation appears to be poor.

Fig. 2. Rack and Gear Installation on Gate

This device was designed to facilitate adjustment of the gate through which the lime moves onto the feeder belt.

The temperature of slaking is controlled by temperature regulators. These consist of thermostatically controlled valves on the line carrying the slaking water. The thermostatic element, encased in a stainless steel tube, extends about 18 in. into the first slaking compartment, where changes in temperature due to variations in lime feed or composition first take place. Temperature recorders have also been installed on each slaker. These are very useful to the operators when putting a cold slaker in operation or making a major change in the feed rate. They are easier to read than the usual thermometers, and provide a valuable record of the operation of the slaker.

The heat exchangers furnished with the three new slakers for the water plant extension consisted of coils made up of copper tubing and fittings fastened to a steel stiffening plate. These coils invariably wore through from the outside, or slurry side, in about 6 months. After replacing them several times, it was decided to replace the copper coils with steel plate coils.

Plate coils are heat transfer units, consisting of two sheets of No. 14 cold-rolled steel with channels embossed in them. When placed face to face and spot- and seam-welded, they form compact and lightweight units, through which the slaking water is circulated to absorb heat from the lime slurry. In the 5 years since these plate coils were installed, no replacement has been necessary. The slaker manufacturer has since adopted the plate coils as standard equipment.

Shortly after the original water conditioning plant was placed in service, it was found that the amount of sandy grit and clinker in the slurry was making it necessary to take each rapid mix and flocculator basin out of service about once a month for cleaning. The

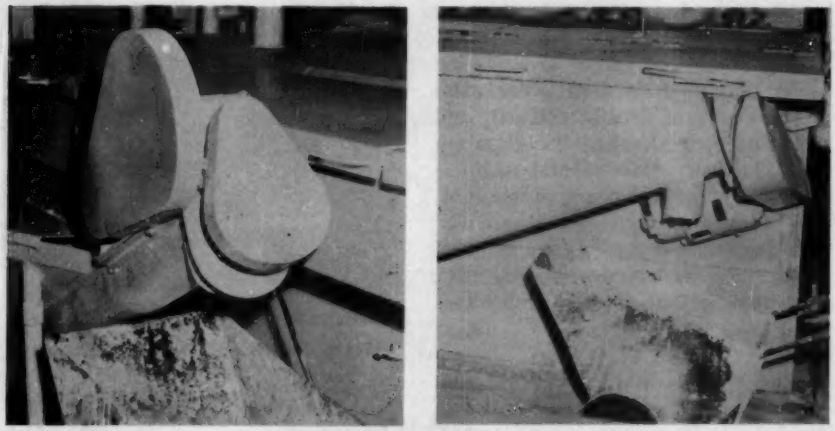

Fig. 3. Grit and Clinker Removers

On the left is the model designed at the Lansing plant, based on data from a manufacturer's experimental model. On the right is the newer model, furnished by a manufacturer.

slaker manufacturer was consulted and very shortly shipped an experimental grit and clinker remover for testing. This design was not the answer to the problem, but enough information was obtained to enable the engineering department to design, and the mechanics at the pumping plant to build, clinker removers that work very satisfactorily (*3*).

When the water plant was expanded, it was stated in the specifications for the slakers that "provision shall be made for continuous removal of grit and clinker from the slaked lime and the slaked lime slurry leaving the slaker." The manufacturer made this equipment an integral part of each of the three slakers, and grit and clinker removers are now standard equipment on most of the slakers of this make.

Another innovation should be noted here. In the grit and clinker removers the rejected material is thoroughly washed before it is discharged to the clinker cart. This has prevented waste of lime, and also the material is drier and has much less tendency to cake and harden in the large clinker hopper into which it is dumped. The use of these clinker removers at Lansing has reduced the cleaning of the primary basins from once a month to once every 3–4 months. Figure 3 shows both models used at Lansing.

After several years of operation, leaks began to develop in the slaker tanks. Inspection revealed that the bottom curved plates and the end plates in the first compartments had been worn very thin by the abrasive action of the grit in the lime slurry, and in places these plates were worn completely through. New plates were welded in place, and then wearing plates were tacked to these plates. These wearing plates can easily be replaced when inspection shows that they are badly worn.

One design feature of the slakers could be improved. The insulated covers on the top of the slakers corrode rather rapidly. The covers are made of 18-gage steel and are of welded construction, being hollow and filled with insulation. Condensation collects inside and is held by the fiber glass or rock wool used for insulation. This causes rather rapid corrosion from the inside. Since it does not seem possible to keep the condensation out, stainless steel construction would be more economical in the long run. Also, the covers should be more satisfactorily counterbalanced, since they are quite heavy and in their present condition are a safety hazard.

Pellet Lime

The use of pelletized lime from the new calcining plant at Lansing (*4*) has presented some new problems to the operating personnel. This pellet lime, because it is similar to small marbles or shot, is easy to convey, and in feeding it forms an even ribbon of material on the feeder belt. The screen size of this pellet lime is quite constant, and for this reason the adjustable gate above the belt seldom has to be readjusted. The pellets are quite hard; therefore, the amount of fines developed by conveying and handling is negligible, and the material never arches or "rat holes" in the bins. Because of its higher density, however, and the fact that a small amount of soda ash has been used as a binder, it must be slaked at a higher temperature than pebble lime. To obtain the slaking temperature required (180°F–185°F), steam lines were installed leading to the first compartment of each slaker. Before a cold slaker

starts feeding pellet lime, it must first be warmed up to about 150°F with steam. Then, when the lime feed is started, the temperature rises quickly to 185°F. When slaked at this temperature, the amount of rejects discharged from the clinker remover is negligible. These same steam lines have proved to be a distinct aid in getting a cold slaker in service when feeding pebble lime also.

Conclusions

Experience at the Lansing Water Conditioning Plant has demonstrated that, for the most part, manufacturers are very receptive to new ideas and suggestions from the users of their equipment. After all, their designers just do not have the opportunity to get day-to-day experience with their brain children, but if the actual users, who have to live with this equipment, let them know of shortcomings or of possible improvements, they will give these ideas every consideration. Many of the improvements that can be made on equipment of this nature seem to be very minor in character, but added together, they improve the operation enormously and make for ease of maintenance or reduce the amount of maintenance very appreciably.

The specifications for this equipment should be very carefully written and worded so that the successful bidder must supply just exactly what is required. Also, although it may seem more economical to install a lime feeder and slaker that will have a capacity great enough for the maximum expected summer plant load, it is better from an operational standpoint to have several smaller units. This gives the necessary flexibility to permit outage time for preventitive maintenance and also provides the extra standby equipment for the high loads. When operated at close to the rated capacities, the temperatures are easier to maintain, and when changes are made in the feed rates, the time lag before these changes show up in the treatment is shortened. If for some parts of this equipment corrosion- or abrasion-resistant materials seem desirable, that fact should be included in the specifications. First cost for some of these items may seem high, but if they eliminate excessive maintenance and improve operation they are actually economical.

Acknowledgments

All of the improvements in lime handling, feeding, and slaking at the Lansing Water Conditioning Plant are the result of the forward looking approach of Claude Erickson, head of the mechanical engineering department of the Board of Water and Light, who wrote the specifications for the equipment, and in the combined efforts of this department, the manufacturer, Marshall Burt, the assistant plant engineer for the water conditioning plant, and the plant operating personnel.

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Factors Affecting Filtration Rates

Herbert E. Hudson Jr.

A paper presented on Sep. 26, 1957, at the Michigan Section Meeting, Detroit, Mich., by Herbert E. Hudson Jr., Partner, Hazen & Sawyer, Engrs., Detroit, Mich.

THERE has been so much discussion of filtration rates in recent years that a review of the various factors affecting the establishment of proper rates seems desirable. Principal factors affecting filtration rates are the quality of the raw water, the quality of the pretreatment facilities and their management, sand size, bed depth, head conditions, and hydraulic conditions in the filter piping.

The original work on rapid sand filters was carried out at Louisville, Ky., and led to the adoption of rate controllers 60 years ago as a means of governing the input and output flows from the treatment facilities and filters, so as to keep the flows in balance. These controllers are also used to limit the filtration rate in the plant. Experience throughout much of the United States has established that water of good quality can usually be produced without undue difficulty by filters operated at a 2-gpm/sq ft rate, using sand of 0.5-mm effective size, a thickness of sand bed of about 2 ft, and an 8-ft loss of head.

Limiting Conditions

The design of filters to date has been almost exclusively based on practical experience, which has demonstrated that satisfactory filter runs can be obtained, with usual design and pretreatment, during those times of the year when filter runs tend to be bothersome.

As in every other engineering problem, some factor of safety must be left in the design. Filtration rates must not be increased to the point where the filters barely prevent the production of turbid water, nor must they be increased to the point at which short filter run troubles get to be extreme. Both short filter runs and passage of flocculated matter through filters are forces that should tend to hold filtration rates down. The forces that tend to increase filter rates are economics and the improvement of pretreatment methods of water. Today, when far better settled waters are being produced in plants of recent design than in plants of earlier vintages, it becomes possible to run filters at increased rates.

In a number of locations it has been observed that, with the usual pretreatment facilities, and filter design as outlined above, trouble occurs at times of weak flocculation, when coagulated turbidity penetrates through the sand beds towards the end of the run. In such places, best practice has suggested washing the filters at lower terminal head losses, although the same result could be accomplished by substituting finer sand, operating at lower filtration rates, or using thicker sand beds. It is hard to change the sand in the filter from season to season, so the usual method of meeting difficulties is to wash the filters at lower head losses,

leaving the filtration rate somewhere near the design point.

Not all waters cause these filtration difficulties, which are notorious for occurring in cold weather conditions when water temperatures are below 40°F. On the Great Lakes, these difficulties frequently occur when the turbidity tapers off after storms and consists of very tiny particles occurring in high concentrations. Treatment conditions usually are not very difficult while turbidity in the water is increasing, since much of the turbidity at that time is coarse material, which is easily flocculated.

This subject has been quite fully reviewed in a number of publications, describing experiences with weak flocculation conditions at Chicago and Baltimore. Many other places have experienced these difficulties. For example, they occur with Missouri River water and with the James River at Richmond, Va. Flocculation aids are frequently helpful in getting around the difficulty, by toughening the floc during these critical periods, which commonly occur from January through March in the United States. The most common flocculation aid in the past was the activated-silicate type which was somewhat troublesome to manage. Development of continuous silica activators help, and now coming on the market are some of the newer organic materials which serve a similar function, but which are not yet fully tried or evaluated for their toxicity.

One of the difficulties with the choice of filter rates in many places is that engineers and operators tend to be guided by general averages rather than by limiting conditions, such as those of weak flocculation. Obviously, the choice of plant design and the plant operation must be based on the most severe conditions. The South District Filtration Plant at Chicago was designed for operation at 3 gpm/sq ft in winter and 4.5 gpm/sq ft in summer. Actually, experimental work has been carried out in this plant at rates continually in excess of 4.5 gpm/sq ft with good success, provided a flocculation aid was used during periods of weak coagulation.

Effects of Filter Materials

It has been established that the ability of the filter sand to remove turbidity is a function of the size of the passages through the sand. Since these passages are of odd shapes, it is awkward to discuss their dimensions, so they are usually referred to in terms of the size of the particles. The technical relationship governing the effectiveness of the sand in removing suspended matter states that this ability of the sand is related to the square of the particle size. Thus, sand of 0.5-mm diameter is twice as good in removing turbidity as sand of 0.7-mm size, and sand of 0.35-mm size is twice as good as 0.5-mm sand.

Although the finer sand will produce a better quality of filtered water, it also produces short filter runs in just about the same ratio as above. If the effective size of the filter sand is halved, the filter runs will be shortened to one-quarter of their former length. Those troubled with short filter runs who want to increase the sand size might increase the effective size 40 per cent, thus doubling the filter runs. Such a change may simultaneously produce an unsatisfactory filtered-water quality.

Another important factor governing the size of the opening through filter beds is the porosity of the sand. The porosity is the ratio of the void space within the bed to the total volume oc-

cupied by the bed, expressed as a percentage. The porosity of sand in a filter bed is about 43 per cent. Other filtering materials having larger void ratios actually have larger size pores. Angular materials, for example, may have porosities as high as 55 per cent. These angular materials, such as crushed coal or crushed quartz, do a less adequate job of removing suspended matter than materials of lower porosity. Correspondingly, they give longer filter runs. If an engineer uses an angular material of 0.5-mm effective size in order to get longer filter runs as compared with 0.5 mm sand, then he must expect lower quality filtered water. The only way of cheating on this relationship is to use some of both. For example, use of a relatively thin layer—4–6 in.—of coarse crushed coal on top of the sand bed of 0.5-mm size works out extremely well. The coal serves as a roughing prefilter, relieving the sand of much of the load, and filter runs are greatly lengthened. This gives the advantage of the low clogging rate of the coarse material together with the high filtering ability of the finer material below it. Apart from methods like this, however, there is no way of getting longer filter runs without sacrifice in water quality except by reducing the amount of load placed on the filters, by decreasing the filter rate or by providing better pretreatment.

Bed Thickness and Rates

Bed thickness is equally important in the behavior of filters, since the thicker the bed is the more complete the removal of material will be. General practice has shown that 2-ft beds do a good job. Beds of 3-ft thickness do a better job, but they are expensive to build and, therefore, are uncommon in the United States. In European practice, however, depths as great as 6 ft are sometimes used. Because of the greater depth used, it is possible to use coarser material, the result being longer filter runs. All these factors are related and affect each other.

Strangely enough, the thickness of the filter bed has practically no effect on the lengths of the filter runs. The author once operated a series of four filters, having thicknesses of 6, 12, 24, and 36 in., respectively. They all had the same size of sand and were operated at the same rates of filtration. The filter run lengths were practically indistinguishable. Changing the sand bed thickness is not going to make much difference in the operation of the filters, but it will make a real difference in the water quality produced. A filter bed twice as thick as another bed will do twice as good a job of protecting the public against unsatisfactory water quality.

Similarly, the higher the filtration rate is, the shorter the filter runs and the worse the water quality will be during critical periods. More than twice as much turbidity in the effluent of a filter operated at a 4-gpm rate can be expected as in the effluent from one operated at a 2-gpm rate. Those fortunate people whose plants always produce perfect water may be able to stand some increase in filtration rates, but an increase in rates must be expected to shorten the filter runs at least in inverse proportion to the rates. Some experimental work has indicated even more marked effects than this, although the evidence on this point is contradictory.

Loss of Head

Another important factor is the loss of head through the filter bed. Hulbert and Feben (*1, 2*), in some out-

standing studies, found that filter run curves at Detroit were practically straight lines from the start of the run onward. This frequently happens in many plants, but others find that the loss-of-head curve, when operating at a constant rate of filtration, curves upward sharply. This refers to strip charts, rather than circular ones. It is rather difficult to see what the relationship is on a circular chart, although it will frequently show what is truly an accelerated rate of increase of loss of head toward the end of the run, which is masked by the shape of the chart. Because of the fact that the time versus loss-of-head curve tends generally to get steeper towards the end of the run, there is not much advantage in operating filters to a high loss of head. About two-thirds as much water will have passed through a filter operated to a 5-ft loss of head as will have been filtered by running to a 10-ft loss of head, sometimes more. Increasing the terminal head loss, therefore, does gain less than might be expected in filter run lengths, but it has a profound effect on the filtered-water quality.

It may be fortunate that most open-bed plants have a limiting freeboard in filters and clear wells, which fixes the maximum loss of head. Pressure filters do not have this protection, however, and thus are more subject to abuse. At one plant, pressure filters were operated with reciprocating steam pumps on both the influent and effluent. The pumps worked in and out of phase and the only good feature was the high loss of head in the underdrain system, since the rate controller diaphragms had been unable to cope with the hammering and had ruptured.

The total loss of head to which the filter is operated is one of the most critical factors in determining the quality of the filter water, since the solid material in the applied water continues to penetrate deeper and deeper into the bed as the loss of head increases. As mentioned earlier, this finally results in breakthrough, at which turbidities may jump from their usual low reading (0.1 units) to as much as 10 units. This is why, at times of weak flocculation, filters are washed at lower head losses in order to obtain a good quality of water.

Most filtration plants are designed today with the clear well maximum level at 10 ft below the settled-water level, thereby eliminating the saving of head by washing filters at low head losses. There is some reason to think that clear wells might well be designed with their ceilings located at about the settled-water level so that it would be practicable to operate filters to low losses of head, and thus gaining some additional storage capacity and saving on pumping costs. It is very difficult to do this with an existing plant, but new ones can sometimes have this feature designed into them. The added cost of the taller reservoir may be offset by the savings in pumping costs.

In an existing filtration plant, there is not much to be done about the size of the sand, its porosity, nor the bed thickness, but controls can be applied to the rate of filtration and the loss of head. These can readily be altered by the operator, although he should use caution. Actually these two factors have been found to be more closely related to each other than many people think. When a filter is operated at a constant rate, as the bed clogs up, more and more of the empty space in the sand is occupied by solids strained out of the water. This means that the water is forced to go faster through the

remaining space to maintain the same total flow. The velocity inside the sand bed is the important item. During the filter runs, it increases as the loss of head increases. If this velocity gets too high, sediment is transported clear through the sand bed. Relationships have been worked out by which the limiting value of this velocity can be determined for a given plant. Thus, by carefully limiting both the flow and the loss of head, it is possible to operate filters so that they will at all times produce safe water (provided that the pretreatment is of good quality). With a system of that nature, higher filtration rates are possible, particularly at lower losses of head.

Filter Bed Conditions

All the foregoing has applied to clean sand beds. Sand beds that are fouled with mud balls and clogged places cannot be expected to do a good job of clarifying water. The surface wash does a very good job of completely eliminating these troubles. Surface wash should be an essential part of the design. Without a surface wash, a good deal of effort and high-pressure water applied through hoses is necessary to keep filter beds in good conditions. When filters are kept in good condition they will perform in accordance with the relationships described. When they are laden with accumulated mud that is not removed by backwashing, cracks through the filter beds, the pulling away of the beds from the side walls, and bed upsets completely destroy the effectiveness of the filters.

Improperly designed backwashing facilities, which may produce uneven wash water distribution, will also upset the performance of filters very badly by causing a mounding of the gravel, sometimes to the point where gravel projects upward through the sand. Under circumstances like this, it is impossible to produce good water. Such filters have to be rebuilt to safeguard the public health.

Hydraulic Conditions

Another factor that severely upsets the operation of filters is unsteady flow. Filters that are directly connected to pumps which may produce variance in flow through the bed cannot be expected to produce good water.

There is one filtration plant which has an improperly designed master control system that maintains the general average desired rate of flow by turning all the filters off and on every few minutes. Obviously, this is rough on a hard working filter, and while the bottom of the reservoir can be seen through the sight well, there is grey haze in the water around the submarine light in the reservoir.

John R. Baylis has discussed another force that has a profound effect on filtered-water quality: the short-time surges in rate of flow through the filter bed caused by peculiar hydraulic conditions in the filter effluent piping and equipment. Such surges may be produced by improperly functioning rate controllers, or by air leaks in rate controller stuffing boxes, which cause the effluent pipe to serve alternately as a siphon and as an open channel, thus causing wide variations in the rate of flow from one minute to the next. This problem can be avoided by locating the rate controller at an elevation below the water level in the clear well, but in an existing plant, this may be impossible to change.

Most filter rate controllers, properly maintained, do a good job, although there is some evidence that the short-tube venturi meter used in them may

produce pulsations in flow. Experimental work carried on in the Chicago Filtration Plant showed severe injury to water quality caused by pulsations resulting from the venturi meters.

Water Quality Standards

Aside from interest in how filters function, there should be concern about the quality of water that they are producing and the methods of measuring it. Of course, the standard bacterial analysis is of vital importance in establishing whether the water produced is always of high quality. Bacterial measurements, in these days of prechlorination, do not give a very good measure of filter performance. About the only measure that tells how the filters themselves are performing is the clarity of the filter effluent. There are several ways of measuring this. One of the handiest methods is through use of the filtered-water sight well, which, when equipped with suitable lighting, lets the operator evaluate the filtered-water quality quickly, easily, and with great sensitivity.

Other methods of checking on filtered-water quality involve measurements with various types of turbidimeters. The Baylis turbidimeter is good to about 0.1 turbidity units, but it is a little bulky and awkward to use. Other turbidimeters are somewhat handier, but considerably less sensitive, and do not ordinarily give precise readings below 1 turbidity unit. They are excellent instruments to use for raw and settled waters and avoid the necessity for preparation of standards. The St. Louis turbidimeter is a very useful one. It uses Nessler tubes, and permits a measure of either settled- or filtered-water turbidities to as low as 0.05 turbidity units. Of course, there is the necessity of preparing standards for it, and new ones are needed at least once a week.

When routine measurements are made with an instrument like the Baylis or St. Louis turbidimeter and records of these measurements are kept, it is relatively easy to find out when the breakthroughs of the filters occur. Of course, an easier method is to watch the sight well, but there are no quantitive figures obtained from that observation nor indications of what the individual filters are doing. The preservation of records of accurate filtered-water turbidity determinations is of highest importance in establishing whether or not filters are functioning properly. It is one of the handiest methods, also, of determining whether or not filtration rates can be increased. If these observations of filtered-water clarity prove that there are never any breakthroughs in the plant, then there is good evidence to think that higher filtration rates might be practicable, particularly when the water quality is good, as it usually is in the summer, when flow requirements are highest. Without such accurate and complete records of plant operations, there is a grave danger of turning loose unsatisfactory water to the public.

As a rule, the public cannot tell the difference between water of 0.05 turbidity unit and that of 1 turbidity unit, except a few people who use tub baths or swimming pools and keep them particularly clean. But the difference between these two qualities of water is of extremely grave importance at those times when there are chlorinator difficulties.

Practical Applications

To find out just what can be done with the filter rates at a given location, it is necessary first to review the de-

tailed records of filtered-water quality. If there are no detailed records, the plant personnel should begin obtaining and keeping them.

A detailed review of filtered-water clarity records tells when and under what conditions breakthroughs occurred. Analysis of the head losses and filter rates at these times, together with data on the sand bed thickness and sand size and porosity, makes possible an evaluation of what may be done at the given plant. This sort of evaluation was made to check the sizing of the sand for the new filters at Detroit's Springwells plant. The analysis verified the conclusion that had previously been obtained from actual trials for the Northeast plant in Detroit.

The same kind of analysis has been made for the Wayne County filtration plant design, and the sand size, bed thickness, and other characteristics have been established from this analysis. The idea of linking the rate of flow and loss of head together by a special filter design has obvious merit. Such variable-rate operation makes possible the savings of 5–10 ft of head in the plant, while maintaining a design that cannot produce unsatisfactory water.

The Wayne County filters are rated at about 2.5 gpm/sq ft for constant-rate operation, but they can also be operated with higher rates of flow in the early part of each run and with lower rates as the run continues. Trial operation of a variable-rate filter is now going on in one of the water plants in the Detroit area, with good results so far. More complete data will be required, particularly with careful observations made during periods of weak flocculation, before the evaluation of the process can be final. So far, parallel filters with constant-rate and the variable-rate operations are producing equal results, as should be expected. The difference in performance should show up when weak flocculation occurs, at which time the variable-rate unit should steadily produce a bettter quality of water.

Conclusion

The problem of filtration rates is a difficult one because so many factors enter into it. The relations between these factors are relatively simple, however, and it is possible to determine just what filtration rates are safe to use at any given location, provided measurements are accurate and records are complete and cover a period long enough to assure that the critical limiting conditions have been included.

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Nebraska Law Affecting Water Resources

Robert Perry

A paper presented on Apr. 25, 1957, at the Nebraska Section Meeting, Lincoln, Neb., by Robert Perry, Attorney, Perry, Perry, and Nuernberger, Lincoln, Neb.

ALTHOUGH water is one of the basic necessities, most people give little thought to rules affecting its use. This has been particularly true of the rules relating to underground water. Because Nebraska is classified in part as a semiarid state, a considerable amount of water litigation resulted from the use of the streams, lakes, and other ground water within the state. There was, however, little law with reference to underground or percolating water.

For the purpose of classification according to legal treatment, there are at least six different types of water found in the state of Nebraska: [1] navigable streams, [2] streams, [3] lakes, [4] ground water, [5] percolating water, and [6] underground streams. The law in regard to navigable streams, streams, and lakes has been defined in considerable detail, both by legislation and case law. The use of underground water for irrigation did not commence in earnest until the state legislature, in 1951, refused to pass the bill which would have permitted diversion of Platte River water out of that watershed. At approximately that same time, several of the natural gas pipeline companies and the electric power groups became aware of a substantial prospective market and attempted to move to make cheap power available.

With hundreds of new irrigation wells going down each year, there has arisen considerable concern as to what happens when this supply of water runs out and concern as to how best to regulate the supply so that it will not be unduly depleted.

Unfortunately, the laws with reference to riparian rights and to the system of prior appropriation of water as these theories are applied to navigable streams, streams, and lakes have little or no application to underground water. The fourth classification, ground water, has reference to surface water not following any well defined channel or stream, and the fifth classification, percolating water, has in many jurisdictions a close relationship to the fourth.

The rule applied to ground water is that it is owned by the person upon whose land it is found at the moment, to use or permit to run off as he wishes, but that the owner cannot artificially repel it from his land to another person's damage, and he cannot artificially control its course of discharge onto another person's land, to the other's damage. With respect to surface water, most of the courts have held that the right to use such water while it is upon a person's land is absolute to the same extent as the right to use any other element or portion of the soil.

Percolating Water

With regard to percolating water, a large majority of the jurisdictions have followed the same rule as for ground water. This rule, in fact, has its origin in early common law decisions in England.

It can well be imagined the tremendous difficulty that the legislation might have in regulating the underground water supply, usually referred to as percolating water, if this rule obtained in Nebraska. If the title to the percolating water were absolute in the owner of the land under which it lay, the percolating water found under land could be used to a point of complete exhaustion, regardless of what that might do to his neighbor's supply. Similarly, if this were the case, the legislature would have no power to regulate the use of water as it would be a vested property right. One decision said, of percolating waters:

> Their ownership consists in the right of the owner of the land to capture, control, and possess them, to prevent their escape, if he can do so, from his land, and to prevent strangers from trespassing upon his land in an effort to capture, control, or possess them. If percolating waters escape naturally to other lands, the title of the former owner is gone. While the landowner may prevent the escape of such waters from his land, if he can do so, he has no right to follow them into the lands of another, and there capture, control, or reduce them to possession.

Of course, even this right of absolute ownership has its limitations in that a landowner cannot pollute water to another's damage. One decision (*1*) states: "One who collects injurious or offensive matter upon premises which by percolation, transmissions, or subterranean streams, or otherwise pollutes his neighbor's well, is liable for the damages thereby sustained."

Fortunately, for the ability of the government to control water use in the state of Nebraska, the supreme court of this state has seen fit to find that there existed a modified water doctrine. In the case of *Olson* v. *City of Wahoo* (*2*), the court stated:

> The English or common law doctrine . . . is that percolating waters are regarded as belonging to the owner of the freehold, like rocks, soil, minerals, and, in the absence of malice, the owner may appropriate such waters while they are upon his premises, regardless of the fact that such use cuts off the flow of such waters to adjoining land.
>
> The American rule is that the owner of land is entitled to appropriate subterranean waters found under his land, but he cannot extract and appropriate them in excess of a reasonable and beneficial use upon the land which he owns, especially if such use is injurious to others who have substantial rights to the waters, and if the natural underground supply is insufficient for all owners, each is entitled to a reasonable proportion of the whole, and while a lesser number of states have adopted this rule, it is, in our opinion, supported by the better reasoning.

Underground Streams

There is also another type of underground water usually referred to as underground streams. The distinction between an underground stream and percolating water is that the underground stream has regular and well defined underground channels, has well defined banks, tunnels, and caverns through which it passes, and it is easily recognizable and distinguishable (theoretically) from a seeping or percolating flow. To these underground streams when properly identified the law of surface streams is ap-

plied, so that in a state where either riparian or prior appropriation is recognized, those same rights may be established in underground streams. With reference to such an underground stream, the owner of a land through which such a stream passes would have the obligation of permitting the stream to flow to his neighbor downstream in a substantially unaltered form, and would have the right to remove from such underground stream only such water as did not adversely affect his downstream neighbors' riparian rights, unless such removal was authorized by a prior appropriation duly established.

The 1957 Nebraska legislature saw fit to pass two legislative bills which take the first small step toward the regulation of underground water. One bill requires that owners of irrigation wells register each well with the state engineer, giving such information as location, date of completion, purpose, capacity, and any additional information the state engineer might require. It also provides that during construction of an irrigation well a log must be kept, containing detailed construction information. The second bill provides spacing regulations for irrigation wells, stating that a minimum distance of 600 ft must be maintained, except in special cases.

It is to be expected that in the near future, when the information available from the registration requirements is combined with the information now in the process of being gathered on the degree and amount of water available, that some future regulation of the use of this water, if found necessary by the legislature, may be expected. Such regulation when it comes will have to find as its hinge for constitutionality the language of the state supreme court in the *Olson* v. *City of Wahoo* case (*2*). In that regard, the future regulation upon use must find as its basis the protection of a reasonable use of the water to all owners. If the regulation requires a degree of abstinence on the part of an owner greater than what reasonable use implies, it must be assumed that the law would then have to provide for an expropriation of the difference by eminent domain.

The great landmark in the Nebraska constitution prohibiting the damaging or taking of private property for public use without just compensation and the language of the supreme court in the *Olson* v. *City of Wahoo* case will serve as the only two well marked points of departure in the development of this relatively new field of law.

References

1. *Beatrice Gas Company* v. *Thomas,* 41 Neb. 662, 59 N.W. 925.
2. *Olson* v. *City of Wahoo,* 124 Nebr. 802, 248 N.W. 304.

Developing a Safety Program for the First-Level Supervisor

Alexander C. Rener

A contribution to the Journal by Alexander C. Rener, Dist. Operations Engr., Water Distr. Div., Dept. of Water & Power, Los Angeles, Calif.

SAFETY in a water utility is no different than safety in a factory, a railroad, or a construction project. To have an effective safety program, the same basic requirements must be fulfilled. A safety program must have the complete and enthusiastic support of management, and all supervisors down to the first level must be made aware of that support. The first-level supervisor must direct the safety program, because top management cannot be aware of the detailed problems that are involved in getting the message to the employees. Accidents are not reduced by management bulletins urging safe thinking. They are reduced by the example and continuous contact of the first-level supervisor with the employees. In a large organization, the safety engineer is of great value in organizing a safety program, in preparing safety material, and in developing safe methods, but the first-level supervisor must be the contact with the employees.

This article discusses in detail the safety experience of a large construction, maintenance, and operation unit of a water utility. The distribution division of the water system of the Department of Water and Power constructs, maintains, and operates the water system within the city of Los Angeles. This includes the installation, maintenance, and operation of all water mains, trunk lines, fire hydrants, meters, services, pressure-regulating stations, tanks, some of the reservoirs, and all other appurtenances dealing with the distribution of water. To perform these various functions requires a personnel roster of 1,200, of which 1,000 are field employees, ranging from district superintendents to laborers. The water system, because of its size of 44 by 28 mi, is divided into five districts, each completely staffed and equipped to perform the above-mentioned functions.

Past Experience

For the past 10 years, the distribution division has had an assigned safety engineer from the accident prevention section. His duties were to visit the various districts and, with a field superintendent, inspect the various construction projects in the interest of safe operation. His position carried no real authority for enforcing safe methods, but his advice concerning safety was generally followed. Upon his recommendation, a variety of safety equipment, such as toe guards, goggles, respirators, hard hats, and gloves, was purchased and their use encouraged. His position and title promoted safety, but no one else took the same interest in the problem. The foreman re-

spected his comments and appreciated the visit to his job, but the safety engineer was only able to see this particular foreman on the job once every 2–3 months. With 110 crews in the field, it was difficult to devote much time to any particular foreman or construction project. Every 2 months a safety meeting was held in each district. This meeting, after work on an overtime basis, was attended by all superintendents, foremen and lead men. A lecture on safety with demonstrations and motion pictures followed by a general discussion constituted the meeting. The men were expected to put to practice the lessons they had learned at the meeting. The safety engineer was doing everything within his power to promote safety and reduce accidents.

After operating in this manner for a number of years, it became apparent that something was missing. The accident frequency and severity rates were static. In spite of the support by management in assigning safety personnel, in addition to purchasing safety equipment and the personal contact of the safety engineer, people were still getting injured at the same rate. In analyzing the safety program, one missing basic element was found, the source of the trouble. The entire program was the personal activity of one man, the safety engineer. He did all the talking and everyone else listened; therefore, there was no effective program, only the personal effort and contacts of the safety engineer, and with 1,000 men, his contacts were necessarily infrequent. We had to find a way whereby his effort and enthusiasm would be utilized and transferred to the first-line supervisor. It wasn't that the foremen and lead men were neglectful, they just did not get enough material out of the bimonthly safety meeting to carry them through the next 2-month period.

Seven-Point Program

After considerable discussion with safety engineers, other utilities and the management of the water system, a new safety program was developed. The safety meetings in the various districts at which the new program was announced were attended by some of top management in order that the personnel would realize the support this program was receiving. Essentially, it consists of seven major points which will be described in detail.

Point 1

The foreman will conduct weekly trench-side or yard meetings on safety subjects. The meetings, held every Monday as soon as possible after arriving on the job, will last 10–20 min, depending on the subject. The accident prevention section will furnish the topics and the necessary information to the foreman.

This point is the most important of the safety program, because it places the entire responsibility of getting the message to the employee on the first-level supervisor. The safety engineer prepares the material, and by the use of mimeographed copies all foremen use the same subject matter. It is the responsibility of the safety engineer to provide interesting safety material written in a form readily understood by the field personnel. Below is a typical weekly safety talk to illustrate the types of subjects which are used.

The subject for this week is "Hoists and Cranes."

1. If we're using a hoist, we'll have *one* man give the signals. Nothing confuses an operator so much as having two or three men trying to tell him what to do.

2. If it's your job to hook pipe, be sure your slings are in good condition and strong enough to do the job.

3. If you are using a clamp, be sure it is free from grease and dirt and in good working order. Make sure it is fastened securely on the pipe, and then fasten the safety so that the pipe won't scissor out.

4. Never swing a load over another man. Stop the operator until every man is in the clear. If men are in the trench, do not bring the load to the edge of the excavation until the men are in a position beyond all danger.

5. Never attempt to catch or hold a length of pipe that appears to be slipping. It's better to have a cracked pipe than a cracked back.

6. If pipe is to be stored along a roadway or street, securely block each length separately. Not only our own men but children at play have been injured because we did not securely block our pipe to keep it from rolling.

7. If, in lowering, we knock out some of our timber, stop proceedings at once and replace the timber. Remember, the man in the ditch must be protected at all times.

8. When lining up pipe in trench, keep your hands away from the ends. *Never* attempt to remove dirt from the pipe until you are *positive* that the hoist man knows what you are going to do.

9. Never stand directly in line with a suspended pipe. It may slide in the clamp or sling.

10. Never take hold of a hoisting cable. You may lose your fingers by having them cut off between the cable and the block.

Any comments?

The language and the sentence structure of the lessons are plain and to the point. The subject, "Hoists and Cranes," is but one from an almost inexhaustible source that is part of the field work in a water utility organization. Many of the foremen read the lessons; others read it over before the meeting and then talk extemporaneously on the subject. Both methods have proved to be effective.

Point 2

The districts will hold five safety meetings a year at the same time as heretofore. The meetings will be devoted to a review of the accidents which have occurred in the past 2 months and a discussion of the weekly topics for the trenchside or yard meetings to be held during the following 8–9 weeks. The discussions at the meetings will be led by a field superintendent, and the safety engineer will act as an advisor and enter the discussions only when specifically requested.

This is a very important point, and by this method the entire proceedings are directed by the line organization rather than by the safety engineer, who is a member of the staff organization. This one point of taking the meetings away from the safety engineer and having a field superintendent take charge had a great effect on the foremen and lead men, making them aware that the promotion of the safety program was their problem. Upon the request of the field superintendent, the safety engineer advises the foremen on the most effective presentation of the material for the weekly meetings.

Point 3

The foreman or the lead man will instruct every new employee assigned to his crew. This instruction will include the handling of shovels and pneumatic tools, correct methods of lifting, and any other crew function in which he will participate. A check list which indicates the instruction given will be filled out and signed by the new employee and the foreman.

This point eliminated a great source of trouble, because it had been the

common practice for many new men injured on the job to claim that they had never been instructed. By this specific assignment to the foreman and a check list signed by the new employee and the foreman, the claim of having received no instruction was eliminated.

it is only casually discussed among the men. Safety consciousness in the crews has developed to the point where the men suggest safety gadgets and eequipment, such as heat-resistant sleeves for use when pouring lead and the cutting of holes in shovels for use when handling wet adobe.

Fig. 1. Crew Examining a Safety Board

Point 6 of the Los Angeles safety program provides for these boards, mounted on the side of each truck.

Point 4

As soon as possible after an accident on the job, the foreman shall assemble his crew and, after hearing the account of the accident, explain to his crew why it happened and how it could have been avoided. The foreman will use these opportunities to develop a safety-conscious crew.

This idea has been very effective, because a group discussion of the accident immediately after it happens always impresses the crew more than if

Point 5

A committee in each district will review each lost-time accident. It will consist of the district superintendent, the field superintendent, the foreman, the man involved in the accident, and the safety engineer. The purpose will be to determine, if possible, why the accident happened and how it could have been prevented. The findings of the committee meeting will be reported on the "Foreman's Report of Investigation of Accident" form.

This point has also made quite an impression on the foreman and his crew. Although the committee is only interested in determining causes and cures, the foreman and his men do not like to be part of it, because it is their lost-time injury that is being investigated.

Point 6

Each crew of four or more men will have a safety board mounted on the truck showing the number of man-days worked without a lost-time injury. The foreman or truck supervisor will change the totals every day.

It is surprising to find the pride that each crew as a group takes in this ever-changing number that is posted on the safety board on the curb side of the truck (Fig. 1). As the man-day figure increases, a rivalry develops between the various crews, and when a man has a lost-time injury, his coworkers on the crew are very much upset. Teamwork has developed to the point where the men take care of each other, watching, advising, and helping to prevent accidents.

Point 7

A safety plaque will be awarded to each crew working without a lost-time injury for 1 year. These plaques will be mounted above the safety board on the trucks.

At the end of the first year, the top management appeared at the safety meetings and presented the plaques and certificates to the foremen. The impression on the men was very favorable, and they appreciated the compliments and pats on the back. The presence of management at the meeting assured the foremen of their continued enthusiasm and support.

Results

The new safety program has been in operation for over a year. From the very start, the first-line supervisors were cooperative and enthusiastic, because they felt that this was a plan that would produce results. In a very short time there was a decided downward change in accident frequency and severity rates, and with management voicing its approval of their enthusiasm, the first-line supervisors redoubled their efforts. The results were gratifying, and the record of the first year shows a 30 per cent drop in accident frequency and a 52 per cent drop in accident severity. In its second year, the effort further to reduce our accident rates is being continued.

This seven-point safety program makes no mention of records and reports, because the procedures for maintaining them have been in operation for many years. An organization starting a safety program should have a separate point dealing with active records and reports, because only by that method can the effectiveness of the program be determined.

A system of bulletins on repetitive accidents, such as strains, sprains, and lead burns, is used to remind the employees of injuries due to carelessness or neglect in the use of safety equipment. Continual repetition by an employee of the same type of accident results in a warning of disciplinary action.

In this type of program the promotion of safety becomes a part of the first-line supervisor's direct responsibility besides giving him experience in leading group discussions. The "tail gate" meeting is not a new development, and the reason it has proved successful in the Los Angeles distribution division is because the foremen

have been given the necessary material at the district safety meetings and are coached by the safety engineer. The cost of maintaining this type of program is no greater than before. It was just a matter of rearranging the responsibilities.

Conclusions

The ideas developed in this safety program may be used in any size of organization. Many water utilities are small and do not have enough employees to warrant having a full-time safety engineer; nevertheless, the material necessary for first-level supervisor meetings is readily available. The AWWA has published a manual, *Safety Practices for Water Utilities* (*1*), which contains a wealth of safety information and program material. By the proper use of the manual at foreman discussion meetings, a very adequate safety program can be organized and maintained. The benefits of safe practices—both humanitarian and monetary—can be derived by a small as well as a large water utility.

If the first-line supervisor is made to feel that the promotion of safety is his responsibility, and if he receives the proper backing and encouragement from management, there is every reason to believe that safety consciousness will become a definite part of every employee's thinking. With that positive attitude, accident frequency and severity rates will be reduced.

Reference

1. *Safety Practice for Water Utilities—M3.* Am. Wtr. Wks. Assn., New York (1956).

Embankment Stability as a Factor in Adequate Sheeting and Bracing

William S. Housel

A paper presented on Sep. 27, 1957, at the Michigan Section Meeting. Detroit, Mich., by William S. Housel, Prof. of Civ. Eng., Univ. of Michigan, Ann Arbor, Mich.

THIS article deals with that phase of sheeting and bracing which is related to the stability of vertical banks or trenches and the forces to which such vertical faces are subjected. It will be further limited to the forces acting on relatively shallow trenches in which the shearing resistance of the soil is sufficient to prevent sliding in the usual sense. Such excavation is involved in the everyday digging of trenches for sewers, water lines, or other facilities. To the uninitiated, such shallow excavations would appear to present no problem; as a result, the great bulk of such miscellaneous digging is done with little, if any, engineering control or supervision. Control and supervision are also frequently inadequate on larger projects where one ordinarily might assume that responsible charge would be vested in construction men of experience, fully aware of the physical forces with which they were dealing.

Relatively shallow trenches or cuts can be deceptively dangerous because the soil may be very difficult to dig and have high resistance to shearing displacement. The fact that the soil may stand unbraced for considerable periods of time contributes to the appearance of safety and, as a result, many excavations in such materials are left without support until a serious accident occurs. Even then, the occurrence often is accepted as an unavoidable accident or act of God, and the demonstration that physical laws have been violated passes with little attention or benefit to future practice.

Scarcely a week passes in any large city without a headline reporting serious injury or, more probably, loss of life in a cave-in. Although no statistics have been prepared for this presentation, it can be noted that this type of accident occurs in every community with such frequency that it is hard to understand why it is not segregated in construction accident statistics.

Use of Model

The study of earth forces acting on any vertical face cut in the soil mass is the first phase in the development of procedures for designing bracing systems for heavier excavations. An understanding of the nature of these forces can be obtained from studies of gelatin models (*1*). Such a device is shown in Fig. 1 where the model has been cast between two glass plates with the vertical face on the left completely supported by a wooden bulkhead. A grid has been etched on the glass plate to assist in studying the form of the failure surface and certain stress patterns which develop when the vertical support is removed.

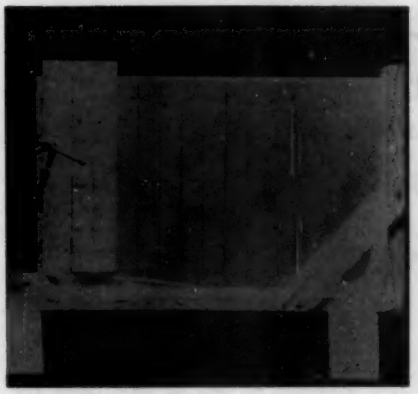

Fig. 1. Gelatin Model Used in Experiments

The squares faintly visible on the glass plate provide a coordinate system useful in the study of the stress patterns the form of the failure surface.

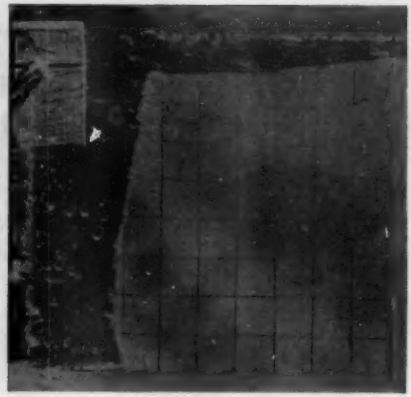

Fig. 2. Model With Support Removed From Vertical Face

Gelatin has subsided, but no failure is yet apparent.

In Fig. 2, the vertical support has been removed and, between the glass plates, the gelatin—which serves as the model embankment—has subsided and bulged considerably although no failure or impending failure is apparent by visual examination.

In making the series of illustrations shown, the temperature was allowed to rise slowly to decrease the consistency of the gelatin mass. Under such controlled conditions, the embankment failure developed very slowly and made it possible to take a series of revealing consecutive photographs. In Fig. 3, the primary slide failure is quite apparent from the separation of a portion of the gelatin mass as a result of tension developed at the top surface. The tension crack shows a depth of about two units of the coordinate system used. As the failure progresses, a portion of the gelatin mass with a width of approximately half the height rotates in a counterclockwise direction about the toe of the vertical face and falls forward toward the simulated excavation. As this takes place, the tension crack deepens and destroys both the temporary tensile force which originally held the mass intact and, along the failure surface, the shearing resistance which had prevented the

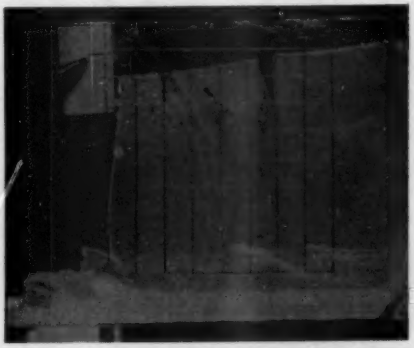

Fig. 3. Primary Slide Failure

Tension crack appears at top, about two units deep.

mass from sliding downward into the excavation.

When the shearing resistance and its associated overturning moment has been destroyed, however, the rotation of the sliding mass stops and the unstable mass slides down the failure surface into the excavation (Fig. 4). Figure 5 shows the condition in which the mass came to rest. The sliding surface itself is also faintly discernible.

One of the reasons for using gelatin in such a model study is that the solidified gelatin has sufficient photoelastic properties that it can be used to obtain an actual picture of the stress pattern that is developed under such a loading condition. While the gelatin was still in the semielastic state, photoelastic techniques were employed to get a series of photographs of the stress pattern. It may be recalled that when polarized light is passed through a stressed transparent solid, interference fringes are developed which form a series of dark or colored bands—depending upon whether monochromatic light or white light is used. Each of the dark bands represents a surface of constant shearing stress. Photographs made of the model in this field of light show the fringe pattern which gives a picture of the actual stresses that have been developed.

Fig. 4. Sliding of Unstable Mass

Action occurs after rotation of sliding mass stops.

Fig. 5. Sliding Mass at Rest

Sliding surface is faintly discernible.

In Fig. 6, the three circular insets are fringe patterns that were photographed in the indicated position and which show the dark band (along a shearing surface) which represents the potential failure surface in the vertical embankment. These photographs indicate the general form of the failure surface which, mathematically, has been found to be a parabola. It also shows that at the toe of the embankment this surface of maximum shearing stress leaves the vertical surface at an angle of approximately 45 deg. Figure 7 is an enlargement of the lower inset at the bottom of the bank, again showing the dark band marking the 45-deg angle of shear failure at the toe of the embankment.

As will be shown, the wedge-shaped portion of the soil mass involved in the primary slide separates at the horizontal surface at a distance from the shoulder of the original mass of half the height of the vertical face or the depth of the excavation. Figure 8, a photograph of a high embankment in a placer-mining operation after this primary slide has taken place, shows the

general contour of the failure surface which still remains. Careful observation of the scars left by such mass displacements reveal the parabolic surface—with a height twice the horizontal distance from a control point—which is a fairly common feature in mountainous terrain where progressive embankment failures have taken place.

shown by a dashed line which makes a 45-deg angle with the vertical face at the toe of the embankment and is perpendicular to the horizontal surface. In order to simplify the mathematics involved, the parabolic failure surface has been replaced by two straight lines, one representing the 45-deg angle of failure on the lower half of the mass,

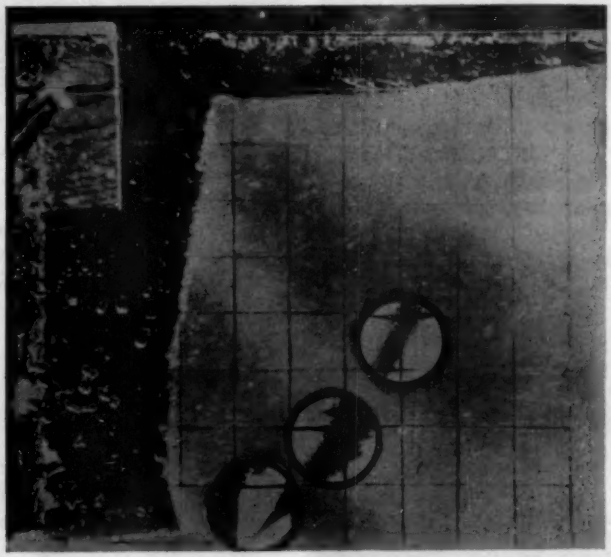

Fig. 6. Fringe Photographs Superimposed on Model

Insets show surface of maximum shear stress.

Analysis of Failures

Attention may now be turned to a mathematical analysis of the types of failure which take place in an unsupported vertical face in a cohesive earth mass. In Fig. 9, a diagram of the wedge-shaped portion of the mass involved in the primary slide or failure is shown. The parabolic surface—which is the true failure surface—is

the other representing the vertical surface on the upper half of the mass. This simplification is convenient in dealing with the forces that are involved in the mass stability and represents an approximation that departs from the more exact mathematical treatment by less than the probable error in evaluating the shearing resistance of the soil and other factors in such a problem.

The maximum shearing stress on the failure surface would occur on an element of mass at A, the bottom of the vertical excavation where the lateral pressure is zero and the vertical pressure on the element is a maximum. The loading conditions at A are such that the principal planes or planes of maximum and minimum normal pressure are the vertical and horizontal planes. Using the well known relationship between principal pressure and maximum shearing stress from the strength of materials, it is calculated that the shearing stress, f, along the 45-deg failure surface is equal to half the difference in principal pressures:

$$f = \tfrac{1}{2}(p_v - p_h)$$

where p_v and p_h are the principal pressures. At A, where the lateral pressure is zero, the shearing stress, which is a maximum, is equal to half the vertical pressure at a depth equal to h—the depth of the excavation:

$$f = \tfrac{1}{2}(wh - 0) = \frac{wh}{2}$$

where w is the weight per unit volume. When this maximum shearing stress is equated to the shearing resistance of the material due to cohesion, the critical depth of excavation or the maximum unsupported height is found—when $f_s = S_c$—to be twice the shearing resistance divided by the weight per unit volume of the soil:

$$h_0 = \frac{2\,S_c}{w}$$

where h_0 is maximum unsupported height and S_c the shearing resistance.

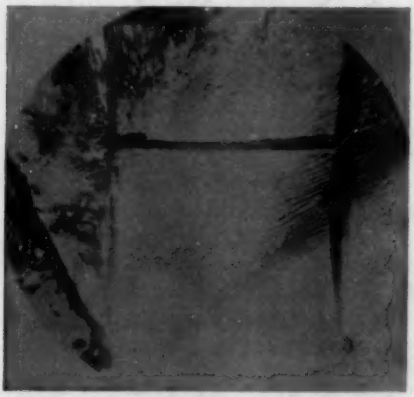

Fig. 7. Enlargement of Fringe Photograph

Shear plane of 45 deg is clearly indicated.

Fig. 8. Typical Failure Surface

View is of a placer mining operation after primary slide has taken place.

It should be noted that the problem under consideration is one of mass stability in a cohesive earth mass or any mass which has cohesion. Obviously, purely granular material will not stand unsupported on a vertical plane or face.

The next portion of the problem (Fig. 9) involves a determination of the horizontal distance, x, at which such a failure surface would cut the horizontal surface of the unsupported mass. In this determination a strip element of mass close to the horizontal surface of the ground where the failure surface is substantially vertical is selected as a free body. The forces acting on the mass at this depth, y, include the pressure of the overlying material which is transmitted directly downward by the element; the shear-

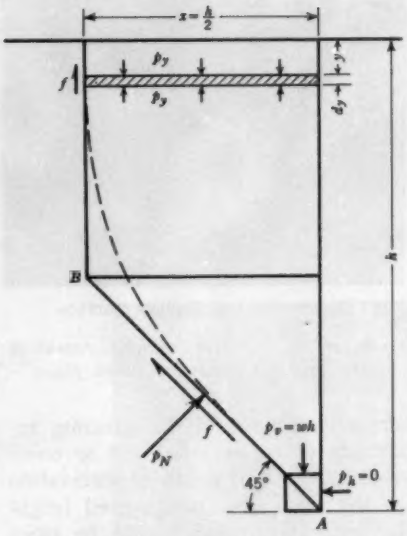

Fig. 9. Diagram of Portion of Mass in Primary Slide

Dashed line shows true failure surface. Toe of embankment is at lower right, horizontal surface at top. Shaded portion is strip element selected as free body for calculation.

ing stress, f, acting at one end of the element; and the weight of the element itself. Equating these forces and solving for the distance, x, it is shown that the horizontal distance from the shoulder or control point to the failure surface of the primary slide—when $f = S_c$—is equal to half the depth of the excavation.

$$wxdy = fdy$$

$$x = fw$$

$$x = \frac{S_c}{w} = \frac{h_0}{2}$$

This relationship between the controlling dimensions of the primary sliding wedge has been verified almost innumerable times in connection with construction operations involving subsurface excavation. It is remarkable how accurately this ratio of 0.5:1 can be observed under field conditions in all types of soils which have cohesion.

Although Fig. 9 shows the general stress conditions in a soil mass adjacent to a vertical face, it does not complete the solution of the problem. Further investigation involves a study of the primary sliding wedge as a free body and a determination of the forces which enter into the static equilibrium of the soil mass. The first step in such an investigation is illustrated in Fig. 10 which shows the distribution of stress along the failure surface when

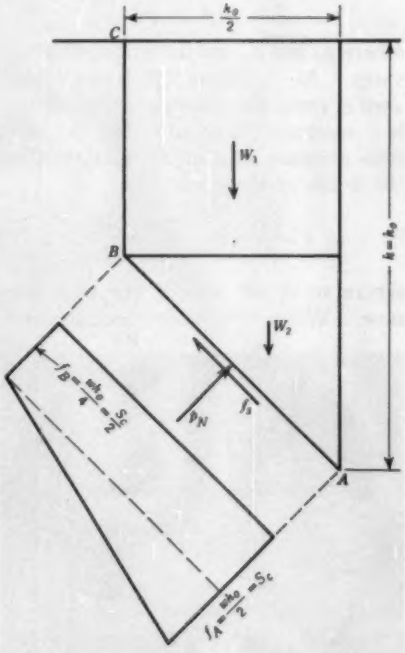

Fig. 10. Diagram of Primary Sliding Wedge as Free Body

Distribution of stress along failure surface is shown. Depth of excavation, h, *is less than or equal to,* h_0, *maximum unsupported height.*

the depth of excavation is less than or equal to h_0, the maximum unsupported height. The shearing stress at A is equal to the maximum shearing resistance mobilized and, as previously indicated, this is half the vertical pressure of the static head of earth. At B the shearing stress is only half the maximum shearing resistance of the soil or, again, half the static head of earth at B. When the lateral pressure on the vertical face of the triangular element is zero, representing unconfined compression of the vertical column, equality of lateral forces on the element requires that the normal pressure be equal to the shearing stress. The average normal pressure, p_N, is equal to the average shearing stress, f_s, which in turn is equal to only three-fourths of the available maximum shearing resistance:

$$p_N = f_s = \tfrac{3}{8} w h_0 = \tfrac{3}{4} S_c$$

Under these conditions it can be shown that the two other equations of static equilibrium involving summation of vertical forces and summation of moments are also satisfied:

$$h_0 = \frac{2 S_c}{w}$$

$$M_A = 0$$

As a result there is no tendency for either vertical shearing stress or tension to develop on the vertical plane BC.

Figure 11 illustrates the forces which develop when the next increment of shearing resistance along the 45-deg shear plane is mobilized. This is accompanied by an increased depth of excavation, h_m, with the full shearing resistance, S_c, brought into play along the plane AB. This additional shearing resistance may be represented in the figure by the shaded triangular area which is added to the shearing resistance and which makes the shearing stress at B equal to S_c. The normal pressure along this face that is associated with the shearing stress is also equal to S_c, making the average normal pressure along that plane uniform, with a value, P_N, indicated as equal to the shearing resist-

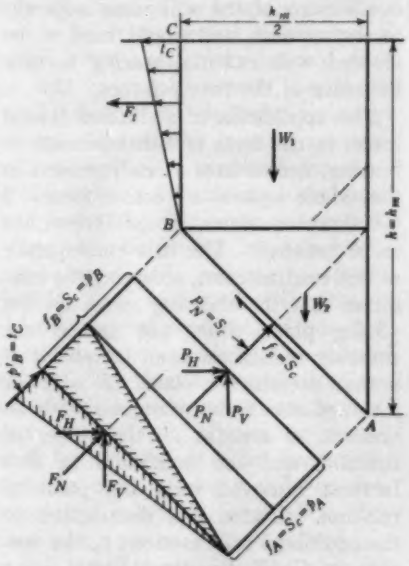

Fig. 11. Diagram of Mass With Full Shear on Plane AB

Shaded triangular area represents additional shearing resistance and normal pressure.

ance, S_c. The first result of the additional shearing resistance represented by the triangular area is to create an overturning moment about A which, in the absence of any bracing, would tend to cause tension on the vertical face, BC. If this tension is assumed to vary according to a linear law, the maximum tension, t_c, would be acting at C. Although such tensile forces

might be resisted in the soil as a temporary condition, it must be recognized that soil is inherently incapable of carrying tensile forces under practical field conditions. Shrinkage cracks caused by evaporation of soil moisture become permanent planes of cleavage which offer no tensile resistance; other structural irregularities may produce the same effect. For these reasons, the equilibrium of the soil mass adjacent to the vertical face would have to be insured with external bracing to take the place of the tensile forces.

The application of additional lateral force, in the form of either tension or bracing, necessitates a readjustment in the whole system of active forces if the three equations of equilibrium are to be satisfied. The first consequence of this readjustment, accepting the condition that the shearing stress on the 45-deg plane could not exceed the shearing resistance, is an increment of normal pressure on plane AB which is assumed also to be triangular with an ordinate of zero at A, the center of rotation, and the maximum at B—furthest removed from the point of rotation. Under this distribution of the additional soil reaction: t_o, the tension at C; F, the total lateral force required in the bracing; and M, the resisting moment about A, are derived in Fig. 11 as:

$$t_o = 0.286\, S_c$$

$$F_t = \frac{0.204\, S_c^2}{w}$$

$$M_R = \frac{0.485\, S_c^3}{w^2}$$

It may also be noted that under the equilibrium of this force system the depth of excavation may be increased to a value designated as h_m which is equal to 2.86 times the ratio of shearing resistance divided by the weight per unit volume of the soil:

$$h_m = 2.86 \frac{S_c}{w}$$

Although the lateral force required in the bracing, its moment about A, and the additional depth of excavation are not large, the example of Fig. 11 is mainly significant as an intermediate set of conditions representing the development of one specific increment of mass reaction involving mobilization of full shearing resistance on plane AB. It is important to recognize that just as soon as the shearing resistance mobilized on this failure surface exceeds that associated with the initial height, overturning moment becomes a factor, causing the tension in the soil at C. Failure to supply an external

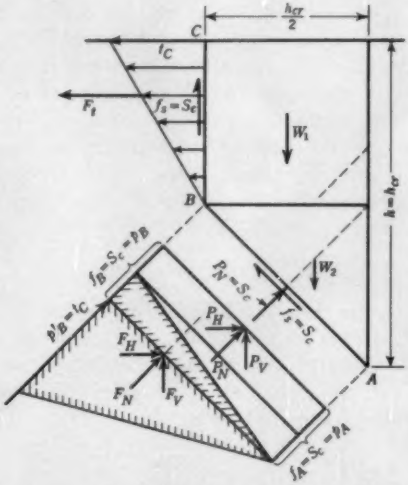

Fig. 12. Diagram of Mass With Full Shear on Planes AB and BC

Shaded area represents additional shearing resistance and normal pressure.

force in terms of bracing to take the place of this uncertain tension invites failure of the embankment through rotation, an eventuality which is quite accurately described by witnesses of many failures of this type. This is true even though the depth of excavation may be relatively small and the shearing resistance of the soil—which would prevent sliding of this wedge—has been only partially mobilized.

Figure 12 shows an example quite similar to that of Fig. 11, with the exception that the magnitude of the forces involved has been increased by assuming that full shearing resistance will also be developed on the vertical plane BC. This represents a limiting case and the maximum resistance that can be realized against downward sliding of the primary failure wedge. Solution of the equations of equilibrium establishes the magnitude of the tensile stress developed along the plane BC, the total horizontal force, the moment which would be required of the bracing, and the maximum depth of excavation, h_{cr}, which can be made without exceeding the shearing resistance of the soil available to prevent downward sliding. Again the solution of the three equations for static equilibrium has been used to establish the limiting values of tension at C, total lateral force, the moment required in the bracing, and the numerical value of the increased depth of excavation, h_{cr}:

$$t_c = 1.72\ S_c \qquad F_t = \frac{2.2\ S_c^2}{w}$$

$$h_{cr} = \frac{5.15\ S_c}{w} \qquad M_R = \frac{9.45\ S_c^3}{w^2}$$

Bracing Requirements

Up to this depth of excavation the lateral bracing that is required could be limited to a single brace near the top of the embankment located at the center of gravity of the tension triangle or the total horizontal force, F_t. A single brace at this point would, of course, have to be accompanied with enough sheeting on the vertical face to distribute this concentrated force without exceeding the local resistance to lateral compression. If more than one brace is used, the center of gravity of the reactions in the brace would still have to be at the center of gravity of the tension triangle if the bracing system was to supply exactly the horizontal reaction, F_t, and the resisting moment, M_R. Other bracing systems could, of course, be used with the center of gravity of the reacting forces any place along the face, but this would change the relationship between horizontal force and resisting moment which, in turn, would affect the distribution of normal pressure and possibly the shearing stress along the failure surface. The magnitude and distribution of the forces necessary to provide mass stability can always be established for any system of bracing which might be used, relying simply on the elementary principles involved in the three equations of static equilibrium. In the solution of such problems, it should be kept in mind that there are only three equations, hence the unknowns involved must be limited to three. This might require either setting the magnitude of the force applied in the bracing and working out the distribution of stress, which would then be imposed on the failure surface, or it may be necessary to make some reasonable assumption regarding the distribution of stress on the failure surface and solve for the force in the braces as has been done in the several examples presented in this discussion.

Conclusion

It seems highly desirable to emphasize that the main significance of the several examples of embankment stability which have been discussed lies in the demonstration that tension is developed in the soil mass under loading conditions which require only partial mobilization of the shearing resistance of the soil. This means that in relatively shallow excavations or trenches the soil mass along a vertical face becomes unstable and may rotate and fall into the excavation long before the tendency to slide reaches a limiting value. Failure to recognize the significance and importance of the forces required to maintain the stability of such a vertical face may account for the failure to supply the necessary bracing in a large volume of minor excavations. One must presume that the frequent occurrence of such accidents can only have resulted from the fact that a great many people who make such excavations or are concerned with them in any active way simply do not realize the elementary facts which have been the main subject of this paper. Under these conditions, it seems to rest upon informed engineers to see that these conditions are brought to the attention of those who may be in a position of responsibility and may be able to prevent such so-called accidents in the future.

Reference

1. HARROUN, D. T. Stability of Cohesive Earth Masses in Embankments. Thesis, Univ. of Michigan, Ann Arbor, Mich. (1941).

B301–57T

American Water Works Association

TENTATIVE

AWWA STANDARD

for

LIQUID CHLORINE

This "Standard for Liquid Chlorine" is based upon the best known experience and is intended for use under normal conditions. It is not designed for use under all conditions and the advisability of use of the material herein described in any water treatment plant must be subjected to review by the chemist or engineer responsible for operations in the locality concerned.

Approved as "Tentative" Oct. 17, 1957

● **Price of reprint—20¢ per copy**

● **Approximate date available—Apr. 15, 1958**

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Incorporated
2 Park Avenue, New York 16, N.Y.

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Tentative AWWA Standard for

Liquid Chlorine

Sec. 1—Scope

This standard covers liquid chlorine for use in the treatment of water, sewage, and industrial wastes.

Sec. 2—Definition

Liquid chlorine is the commercially available form of liquefied elemental chlorine gas. In liquid state, chlorine is amber colored and about 1.5 times as heavy as water. It exerts a vapor pressure varying with its temperature, from 0 psig pressure at −28.8°F to about 150 psig pressure at 100°F. At ordinary room temperature liquid chlorine exerts a pressure of about 90 psig. When the pressure is released, liquid chlorine vaporizes into a greenish-yellow gas, about 2.5 times heavier than air.

Sec. 3—Requirements

Liquid chlorine shall be supplied and shipped in cylinders, 1-ton tanks, or single- or multiple-unit cars as specified. Liquid chlorine shipping containers shall conform to applicable regulations of the Interstate Commerce Commission. The containers shall be reconditioned, maintained, and loaded in strict accordance with the latest edition of "Container Procedure at Chlorine Packaging Plants," issued by the Chlorine Institute, Inc.

Sec. 4—Impurities

Chlorine shall be 99.5 per cent pure by volume as obtained from vaporized liquid chlorine, and suitable for the treatment of water, sewage, and industrial wastes.

When present in excessive concentrations, impurities such as bromine, nitrogen tetroxide, nitrogen trichloride, silicon tetrachloride, and sulfuric acid may cause operating difficulties or dangerous conditions.*

Sec. 5—Sampling

Samples shall be selected for tests from containers as follows:

a. Cylinders. Three per cent of cylinders in each carload or less than carload, but not less than two cylinders, shall be taken at random.

b. One-ton tanks. A sample shall be taken from each tank.

c. Single-unit car. A sample shall be taken from each car.

d. Multiple-unit car. A sample shall be taken at random from at least one unit.

When approved by the contracting officer, a sample of the bulk liquid

* Research work is seriously needed to provide suitable analytical methods for determination of these impurities and acceptable limits for their presence. When such research work has been successfully completed, appropriate limitations on these impurities will be added to this standard.

chlorine in the storage tank used exclusively for filling the smaller containers may be withdrawn and analyzed for lot acceptance prior to the actual transfer of material.

Sec. 6—Inspection

All containers shall be carefully examined and any which show evidence of leakage, damage, or corrosion shall be rejected. Cylinders and 1-ton tanks having a pressure exceeding the vapor pressure of liquid chlorine shall be rejected.

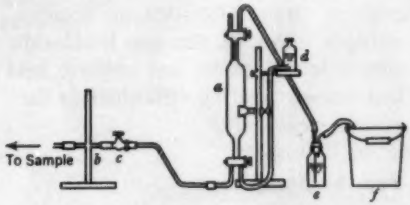

Fig. 1. Test Apparatus for Liquid Chlorine

Letters refer to components described in text as follows: a, *buret;* b, *filtering tube;* c, *needle valve;* d, *leveling bottle;* e, *water bubbler; and* f, *pail with caustic soda solution.*

Sec. 7—Assay Test

7.1. *Specimens.* Test specimens shall be obtained by drawing off chlorine in the liquid form, inverting the cylinder container or using liquid lines for tanks and cars. The liquid chlorine shall be passed through a suitable re-evaporating coil and the evaporated gas admitted to the test apparatus.

7.2. *Apparatus.* The test apparatus shall consist of the following items connected as shown in Fig. 1:

a. A buret such that 100 ml is contained between stopcocks which are 12 in. apart. The upper outlet tubes shall be 1¾ in. long and the lower outlet tubes shall have a total length of 3 in. and shall be bent 90 deg at the midpoint. The stopcock plugs shall have a 2-mm bore, and the outlet tubes a 3-mm bore. The calibrated neck of the buret shall be made of 5-mm bore, thick-walled tubing with a capacity of 1 mil, subdivided to 0.02 ml, and every 0.10 ml shall be numbered. Graduations shall extend at least halfway around the circumference, except that every fifth mark shall extend completely around the circumference. The graduated portion shall conform to US Bureau of Standards Circular C434.

b. A filtering tube consisting of a 6-in. length of ¼-in. iron nipple filled with glass wool.

c. A needle valve with a ¼-in. copper tube extending from it to the bottom of the buret, there joined to one of the outlets by close-fitting rubber tubing.

d. An aspirator bottle used as a leveling bottle. The bottle shall be connected to one of the outlets of the buret by means of heavy rubber tubing 24 in. long. The tubing shall be wired to both the aspirator bottle and the outlet.

e. A water bubbler for controlling the flow of chlorine.

f. A pail filled with a solution of caustic soda for absorbing the chlorine gas from the bubbler.

All connections shall be gas tight. The buret and all connecting systems shall be thoroughly cleaned and dried before testing.

7.3. *Preparation of buret.* The stopcocks shall be given a thin, even coating of chlorine-resisting stopcock grease. The leveling bottle shall be

connected to one of the lower outlet tubes and the bottle then filled with clean, dry mercury. The lower stopcock shall be adjusted to let a little mercury enter the buret, and it shall then be closed. The leveling bottle shall then be raised and lowered several times and the tube collapsed toward the leveling bottle to exclude air bubbles completely. The stopcock shall then be turned to allow the mercury in the buret to be discharged through the other outlet into a beaker. A second leveling bottle shall be attached to the second lower outlet, and the bottle shall be used to flush the buret thoroughly, first with Formula 30 alcohol * and then with benzene. After the buret has been flushed, the bottle used shall be disconnected, the buret thoroughly dried with clean, dry air, and the buret connected to the system.

7.4. *Procedure.* The apparatus shall be connected to the test specimen supply as shown in Fig. 1. The needle valve shall be closed and the stopcocks on the buret opened to allow the gas to flow through the buret to the bubbler. The needle valve shall then be opened very slowly to allow the evaporated gas sample to pass through the apparatus at a rate of ten bubbles per second. Fifteen minutes at this rate is sufficient to purge the buret and fill it completely with the chlorine gas sample. When the gas buret is completely purged and filled with the sample, the lower stopcock shall be closed, the needle valve closed, and the upper stopcock closed. The valve and stopcocks shall be closed as quickly as possible in the above-mentioned order. The buret shall be disconnected from the needle valve and the bubbler by removing the tubing from the outlets. The buret shall remain undisturbed for 10 min at laboratory temperature, and then the upper stopcock shall be opened for an instant to bring the sample to atmospheric pressure. The buret shall then be removed from the ring stand and the lower stopcock opened to permit the mercury from the leveling bottle to enter the buret. The gas shall be kept under a slight pressure by having the mercury in the leveling bottle 2–3 in. higher than in the buret. The buret shall be shaken gently but constantly so that a fresh surface of mercury is exposed to the chlorine. The buret shall be shaken in such a manner that mercury does not enter the graduated portion until the body of the buret is completely filled. When the body of the buret is completely filled with mercury, the buret and leveling bottle shall be attached to a ring stand and the level of the mercury in the bottle adjusted to about the same height as in the graduated portion of the buret. The surface of the mercury in the buret shall be kept moving slightly by pressing the connecting tubing between the thumb and fingers. If no further change in the height of mercury in the buret is noted after 10 min, the liquid surfaces in the leveling bottle and buret shall be brought to the same level and the volume of residual gas accurately noted.

7.5. *Calculation.* The chlorine shall be calculated as follows:

$$\text{Per cent chlorine} = 100 - \text{ml residual gas}$$

Sec. 8—Rejection and Retests

When sample cylinders or units of multiple-unit cars fail to conform with

* A 10:1 ratio of ethyl alcohol to methyl alcohol (US Treasury Department formula).

this standard, tests may be repeated using twice the number of cylinders or units previously sampled. When a 1-ton tank or single-unit car sample fails to conform to this standard, tests may be repeated, using two additional samples from the same tank or car. If samples fail to pass retests, the chlorine represented by samples shall be rejected. Containers filled with rejected chlorine shall not be resubmitted unless full particulars are furnished concerning previous rejections and the measures taken to overcome the defects.

Reducing Traffic Interference During Main Repairs

Byron E. Payne

From a discussion presented on Sep. 24, 1957, at the Kentucky-Tennessee Section Meeting, Louisville, Ky., by Byron E. Payne, Chief Engr. and Supt., Louisville Water Co., Louisville, Ky.

All are familiar with the problem of public impatience when it is necessary to cut a hole in street pavement to install or repair a main or a service. When this problem was aggravated by a recent main cleaning in Louisville, a simple little idea was developed which has partially solved the difficulties to the point that there are now very few complaints about traffic interruptions.

Most of the pavement cuts are from 3 × 4 ft to 4 × 6–7 ft. A group of ¾-in. thick steel plates was purchased ranging from 4 × 5 ft to 6 × 10 ft in size. The edges of these plates are beveled, and holes have been drilled in them for handling. The plates are large enough so that at least 12 in. of pavement is covered on all sides of a cut. To prevent movement, two large flat-headed spikes are driven into holes drilled into the pavement.

In practice, the pavement cuts are made in off-peak traffic hours, the excavated material hauled away, and the repair work accomplished. The hole is then refilled with backfill sand, and the concrete slab is replaced. The plate is then put into place. When the concrete has been properly set, the asphalt paving crew removes the plate, replaces the asphalt, and also replaces the plate. When the asphalt has hardened, a crew removes the plate.

The plates are handled by a power hoist mounted on a truck dispatched from pipe yard headquarters. Since the other utilities in the city are also using plates, each company marks its plates with its initials, bead-welded on the top face.

The idea for the plates was developed by the Mayor's Utilities Committee, a group of representatives from the engineering staffs of each of the utilities, the city Department of Works, and the Traffic Department, which meets once a month to discuss various problems of interest. A very useful spirit of cooperation and a better recognition of all utility problems has resulted from these meetings. An example of another idea developed by the committee is the exchanging of a daily work sheet, which shows the location of each construction crew of each company, so that all utilities can plan work more intelligently with less inconvenience to the public.

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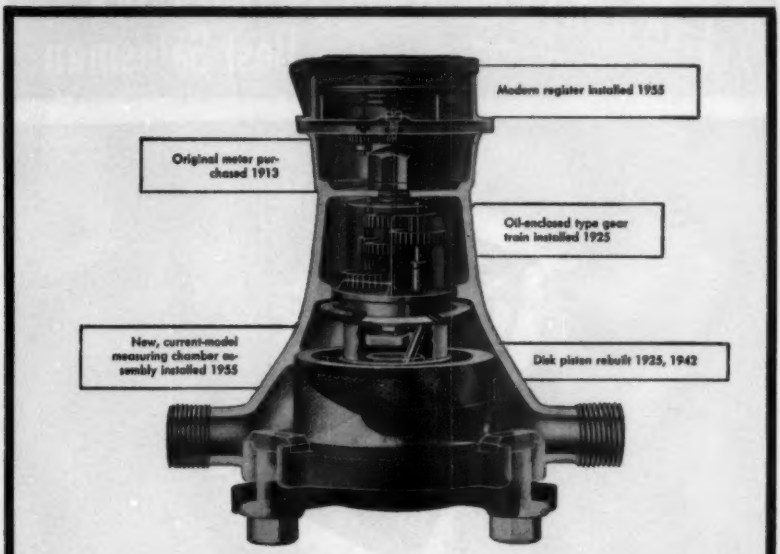
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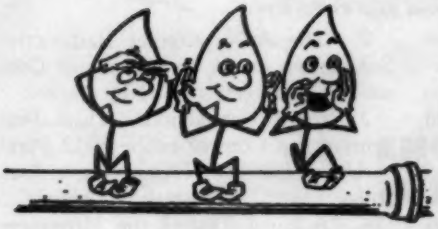

Percolation and Runoff

The honeymoon is apparently over for the engineer; at any rate, by the close of 1957, practically all signs indicated that the end of the postwar "shortage" if not actually on hand was at least in clear sight. Even the fact that it was a drop in demand that brought the end so suddenly in sight does not mean that it is only a mirage, for, as *Fortune* pointed out in calling "The Turn in the Engineer Market" in its December 1957 issue, there was a lot of "water" in industry's demand for engineers that was squeezed out with the leveling off of business activity and the greater availability of engineers resulting from layoffs in the aircraft industry and a reduction in calls to military duty. The fact that industry is less likely now to hoard the engineer doesn't mean that it doesn't treasure him, however, for during 1957 the average starting salary rose to a record $470 per month, and recruiting, though undoubtedly more discriminating, was no less general. Similarly the fact that engineer help-wanted columns declined in length and that fourteen young engineers actually found it necessary to hire a room to recruit employers at the New York ASME meeting in December hardly meant that everyone—and particularly water utilities—had all the engineers required. Nor, fortunately, did the shock of finding an unemployed engineer cause any pause in the efforts of EJC's Engineering Manpower Commission, the Scientific Manpower Commission, the federal science agencies, or Ike, himself, to encourage the training of more engineers and scientists, for the need for engineers is expected to continue to outgrow the need for additional workers in other fields at just as rapid a rate from now until 1965 as it has since the war.

If the honeymoon is really over, however, this may well be the time for water utilities to step in and offer the comforts of a quiet married life. Particularly to some of those who have been the "water" in industry engineer stockpiles, the challenge and responsibility of a job that requires full and full-time application of engineering talents ought to be particularly appealing. This is not to say that we ought to give up the struggle for a dowry that will let us compete with industrial sirens on their own terms. Until we get it, however, the promise of celebrating some anniversaries ought to have a special appeal to the best of disillusioned fortune hunters.

Yoo-hoo! En-gin-eeee-er!

A honeymoon just began in New Jersey, meanwhile, despite the end of

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a shortage, when, on Dec. 27, Governor Meyner signed into law a pair of bipartisan bills that were the first steps of a 2-year legislative program to build new reservoirs that will fortify the water supplies of the highly industrialized northern metropolitan area. This union of Republican and Democrat on the water supply issue following many years of political obstructionism came after the rains that brought an end to this year's crisis of empty or near-empty reservoirs. Despite the reprieve, however, it was apparently obvious to both parties that each new drought has brought them closer and closer to catastrophe—and that catastrophe is poor politics. Before their honeymoon is over, much can, of course, go wrong. But having already picked Round Valley and Spruce Run as the names for their first- and second-born, the two are certainly off in the right direction. And a lot of thirsty North Jersey godfathers will no doubt be ready with their ballots if either sues for divorce.

The 4th EJC Nuclear Engineering and Science Congress will be held at the International Amphitheater in Chicago, Mar. 17–21, 1958, in conjunction with an extensive exhibit titled "Atomfair." This event will afford an excellent opportunity to obtain the latest information on various aspects of atomic energy.

More than 200 papers are scheduled for presentation. Session 35, to take place Friday morning, Mar. 21, will deal exclusively with subjects of direct interest to water utility personnel. The papers tentatively listed for this session are:

1. "Studies of Radioactivity in the Chicago Water Supply"—James C. Vaughan, R. W. Schmidt, Alfred Tenny, and Arthur Shor.

2. "The Adsorption of Radioactive Substances on Waterborne and Consolidated Materials"—F. B. Barker.

3. "Cross Connections: Their Prevention and Control in the Y-12 Plant of Union Carbide Nuclear Co. at Oak Ridge, Tenn."—Gustave J. Angele.

4. "A Field Test of the Movement of Radioactive Cations Through an Underground Formation"—B. B. Ewing.

5. "Radionuclides in Reactor Cooling Water—Identification, Source, and Control"—Dade W. Moeller, George W. Leddicotte, and S. A. Reynolds.

6. "Laboratory Chemical Coagulation Studies on the Removal of Fallout and Radioactivity in Waters"—Lloyd R. Setter and Helen H. Russell.

7. "The Use of the Gamma Spectrometer for the Identification of Radionuclides in Water"—G. Richard Hagee, Abraham S. Goldin, and Conrad P. Straub.

8. "The Sensitivity of Counters for Measuring the Beta Activity of Water"—G. Richard Hagee, Abraham S. Goldin, and Lloyd R. Setter.

9. "Some Methods for the Determination of Radionuclides in Water in Low Concentrations"—Bernd Kahn and S. A. Reynolds.

Down with 'water superintendent,' we said last October (P&R p. 38); down with "telegraph superintendent" echoed the Western Union Telegraph Co. a month or two later—only their echo was louder than our voice for it was in the form of an order retitling all local superintendents throughout the nation "district managers," which the company feels "reflects more accurately the high-level stature of the job."

Rise ye potentates of the public weal, rise ye lords of salubrious liquid, throw the super out!

(Continued on page 46 P&R)

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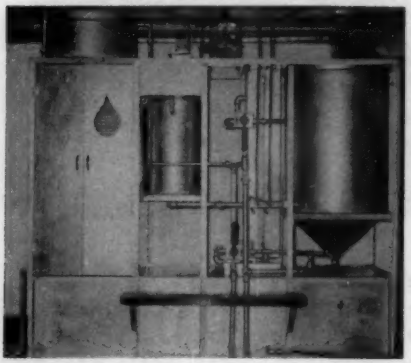
(*Continued from page 44 P&R*)

Do-it-yourself water supply isn't as unusual as it may sound. After all, the 13,000,000 home well supplies, serving approximately 5.4 bgd to suburbanites, exurbanites, and just plain country folk have to be classified as such. Actually, though, it isn't until going down where the water is safe is impractical that doing it yourself really involves some doing. Of course, if you happen to have some hydrogen, oxygen, a jigger, and a shaker around, you can always, as a New Jersey housewife during the recent drought proposed, produce water. As a matter of fact, by using a 2:1 ratio, putting in



a lot of ice, and letting the stuff stand around in a warm room, you *can* usually get a pretty cool drink of water if you time it just right. But for people who don't have the time to go through this procedure for their 60 gpd, two new gadgets have been designed:

First, for those who have a pond, lake, stream, spring, or even an unapproved well within reach, AWWA member Russell Culp of the Kansas Health Dept. has just devised a home water purifier capable of treating up to 1,500 gpd, using chlorination, coagulation, sedimentation, taste and odor removal, and rapid sand filtration. The unit (see cut), which measures $2 \times 7\frac{1}{2} \times 7\frac{1}{2}$ ft and weighs 700 lb empty, is built to sell for from $500 to $800. In removing "bacteria, turbidity, algae, iron, manganese, taste and odor, and other undesirable substances," the semi-automatic device produces a water that will meet the USPHS drinking water standards for bacteriological quality. Operation cost, as determined on the pilot plant now operating in Russ's basement, works out to less than 5 cents per 1,000 gal treated. Tentative manufacturing arrangements have been made, but as yet no sales and service outlet has been found.

Then, for those in coastal areas who have access to sea or brackish waters, Dr. Kenneth C. D. Hickman, working through Aqua Stills, Inc., of Rochester, N.Y., has designed a home vapor compression still capable of turning out 500 gpd of fresh water from sea water. Entirely automatic and no larger than a washing machine in size, this "baby" version of the commercial still that Hickman designed for the Dept. of Interior will sell for from $1,500 to $300, depending upon the quantities manufactured. In turning out a clear, odorless, safe drinking water from the briny, Hickman's "baby" is said to use less electricity than a home laundry.

What with equipment like this available, water utilities will do well to look to their franchises. Given any such good reason as continual sprinkling bans or inadequate pressure, do-it-yourselfers may one day decide to do, leaving the public water system as just another piece of fire department equipment. In other words, do unto customers as they would do unto themselves unless you would that they do unto themselves!

(*Continued on page 48 P&R*)

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(*Continued from page 46 P&R*)

A United Engineering Center, to house the headquarters of the nation's leading engineering societies, is planned for construction in New York City. The building, a 20-story tower surrounded by lower structures and landscaped grounds (see architect's sketch above), will be located at United Nations Plaza, First Avenue between 47th and 48th streets. The Center, with occupancy planned for 1960, will replace the present Engineering Societies Building at 29 W. 39th St., which has been made inadequate by the tremendous growth of engineering in the past 50 years.

In addition to office space, the new building will include the Engineering Societies Library, exhibition space, an engineering hall of fame, and meeting and dining rooms. Also planned are a number of centrally operated services to facilitate the work of the various societies. In all, there will be about 250,000 sq ft of floor space.

In charge of the project is United Engineering Trustees, Inc., a management organization set up by the five founder societies—ASCE, AIMMPE, ASME, AIEE, and AIChE. Eleven other societies, including AWWA, have been invited to occupy the building. The total membership of these sixteen groups exceeds 250,000.

The Center is expected to cost about $10,000,000, half of which, it is hoped, will be contributed by business and industry. An Industrial Building Fund Committee has been organized, with Herbert Hoover and Alfred P. Sloan Jr. as honorary chairman and vice-chairman, respectively. The fund-raising campaign will continue throughout 1958, under the chairmanship of Mervin J. Kelly, president of Bell Telephone Labs.; campaign treasurer is John J. McCloy. The project has been endorsed by many distinguished scientists, educators, and industrial executives.

Architects for the Center are Shreve, Lamb & Harmon Assocs. Engineers for the planning stage are Jaros, Baum & Bolles; for the structural stage, Seelye, Stevenson, Value & Knecht.

A conservation piece, indeed, is the sad story of Mrs. Gladys Gale of Oakland, Calif., whose husband sued her for divorce last December merely because she tried to save some water Actually, it wasn't that Mr. Gale insisted that she waste water, it was just that he wasn't conservation-minded enough to accept water saving as a legitimate excuse for Mrs. Gale's taking a bath with Air Force Sergeant George Holt, even though he knew that "the hot water supply at their cabin in the Mariposa–Big Tree area was 'limited.'" It is understood that there is no limit to the hot water in which Mrs. Gale finds herself now.

(*Continued on page 50 P&R*)

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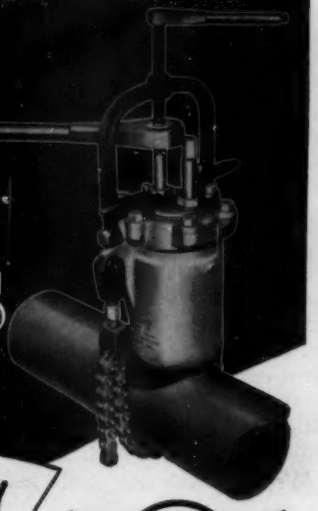

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(*Continued from page 48 P&R*)

The coffee break, which seems more and more to be that period of the working day which is occasionally interrupted by work, has apparently been spreading so rapidly that even our coffee buyers haven't been able to keep up with it. Thus, though the price of a cup of coffee has been boosted at least once since 1949, coffee ain't what it used to be. According to Charles G. Lindsay, manager of the Pan-American Coffee Bureau, "people have watered down their coffee from the rich full-flavored brew they made in 1949, when one pound made about 46 cups, to the weak, brown water of today, which is made at the astonishing rate of 64 cups to the pound."

We, who like our coffee black and our water clear, have long since broken with the break, but, on the whole, as the brew has grown weaker, the demand has grown stronger. That could be because it now requires more coffee to produce the equivalent caffein, of course, but more often, we'd guess, it is time rather than caffein that is of the essence. Meanwhile, just how undeniable the demand can become has been well illustrated on a repair job on the elevated tank at Assaria, Kan. Crew members, who spent 8 hr per day on the job in the tank, insisted on morning and afternoon coffee breaks. As each coffee break involved their climbing out of the tank, down the ladder, walking two blocks to a cafe, drinking their weak, brown water, and

The Harry E. Jordan Scholarship Committee (left to right: N. M. deJarnette, Carl N. Brown, Paul Weir, Robert E. Stiemke, and Daniel A. Okun) are shown at Atlanta, Ga., where they met to choose the 1958 awardee. Their selection was James D. Goff, of Dallas, Tex.

(*Continued on page 82 P&R*)

ALCO *steel mains with high-carrying coal-tar-enameled inside surfaces help New York City distribute over a billion gallons of water per day.*

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Hold That Underline

To the Editor:

Why not underline all illustration references in the text of technical articles? On looking through a journal, you are intrigued by some subject matter and say to yourself, "My, what a purty graph! I wonder what it means?" Or, maybe, it is a table. It seems interesting; the title is nice, but what has the author to say? So you try to find what useful information may be embalmed in Fig. 1 and Table 8. If you read the text, you might as well start at the beginning, and this is slow work. If you focus on capital *F*'s and *T*'s and numerals, a whirly feeling may take over. I just think that Figure 1 and Table 8 are easier to find than "figure one" and "table eight," and that the present custom is not too helpful.

CLARENCE R. HENRY

Miami, Fla.
Nov. 18, 1957

The editorial staff, perhaps much more than the reader, has an interest in being able to pick out these references—particularly in working on the makeup of an article. But despite the very real help that a means of making the references stand out would be, we feel that it is undesirable as well as impractical.

In the first place, the underlining of letters in linotype composition involves the use of special type and expensive hand work. The same effect can, of course, be achieved by the use of boldface, but even the setting of boldface together with the regular typeface of the Journal would require the use of special "mixing" machines for composition. The printer has one or two of these, but it would not be practical to set the entire text of the issue on them. It would be practical to use italic, but the fact that the italic letters are actually lighter would not make the references much, if any, easier to see, and it would make them harder to read. We do find that the use of arabic numerals makes the figure and table numbers a lot easier to spot than if the numbers were spelled out, and, of course, in most articles reference to figures and tables will appear on the same page or quite close to the illustrative material itself, thus making the search not too difficult.

On the count of undesirability, our first objection to either boldface or underlining or even any large amount of italics is an aesthetic one. We try to keep the Journal page as attractive as possible, and the addition of a sprinkling of dark and light type, or underlining that would make the space between lines uneven, would not, we feel, contribute to that end. But we think, too, that we do not want to encourage readers to spot-read our illustrative material, for many of them can be misled by doing so. The "purty graphs" should attract, but we want our readers to go back to the beginning and see how they fit in—or at least to turn to the conclusions and see what the author thinks he has proved.

Despite our disagreement in this case, we could wish that more of our members would give us the benefit of their viewpoints on style and format.—ED.

One More Waiter

To the Editor:

In reading your October issue (P&R p. 50) I was especially interested to note your "Hurry Up and Wait" item as we have one here on *Public Works* magazine, too, though not so much wait. Our I.

(*Continued on page 54 P&R*)

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(Continued from page 52 P&R)

James Barger within the past month or so received a Bronze Star for distinguished service with the 13th Armored (Black Cat) Division in the ETO back in 1944.

W. A. HARDENBERGH

New York, N.Y.
Nov. 30, 1957

In congratulating colleague Barger, we wonder if we have misread significance into the fact that it took just 13 years for the 13th to honor a Black Cat.—ED.

HRH for HOH

To the Editor:

Your story of "HOH for HRH" in December's P&R (p. 54) caused me to do some thinking on this matter and to conclude that there is nothing wrong with the water plants of Ottawa or Toronto and that, if the Queen were actually a visitor at the plants in these cities, her security officers would permit her to drink the water in question. But one must realize that even with good safe waters, there is room 'twixt the faucet and final recipient for trouble; thus, the Queen's security officers must have insisted that sealed bottles of water be brought to the table and opened there, not to take any chances.

The Ottawa and Toronto water plants are not set up to bottle water, so they would naturally have to appeal to some bottling plant such as Coca-Cola or Canada Dry. These plants would be using Lake Ontario water *after it had already been taken from the lake by the Toronto water works, coagulated, filtered, chlorinated, dechlorinated, etc.*, and was in fact a normal city water; but the bottling plant would put it through its secondary filters, containing carbon, to make sure that it had no residual turbidity or residual tastes and odors. I doubt that all this was done in regard to the bacteriological safety of the actual city waters but was really done to prevent any anti-royalists from putting something into the water after it had left the water tap.

All of the Toronto soda pop people use Toronto city water. Coca-Cola throughout the world always uses local city waters but, if in doubt about the "bugs," rechlorinates and refilters. Here in Brantford Coca-Cola gave up rechlorination due to the great sterility of our water but does soften and does refilter just to make sure that no stir up of the mains will cause turbidity. I am sure the Queen drank Toronto city water after it had been merely put through secondary filtration at a bottling plant, bottled, and then officially sealed—just for table-side security. Back home in Buckingham Palace, I imagine that if she were in the kitchen she would just grab a glass and go at it the way that you and I would, and I am sure that her husband and kids would do the same thing. In this instance I am sure that the Canadian security guards were protecting their own reputation more than they were protecting the Queen.

Meanwhile, Queen or no Queen, she is a very nice girl and she stands very high among us water works men in that she at least drinks the stuff that is essentially our bread and butter. She is always on hand to open up new water works schemes in Britain, and her husband is an honorary member of our British opposite number, the Institution of Water Engineers, a group that he is constantly identified with, due to his interest in science and industry. So, as AWWA members, we can count them on our side.

D. B. WILLIAMS

Brantford, Ont.
Nov. 7, 1957

If Don Williams' speculations are correct, our concern about the lack of disinfectant is cleared up. And we certainly agree with Don that a queen who appreciates water is really a queen!—ED.

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Key: In the reference to the publication in which the abstracted article appears, **39**:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, **39**:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH—Bulletin of Hygiene (Great Britain); CA—Chemical Abstracts; Corr.—Corrosion; IM—Institute of Metals (Great Britain); PHEA—Public Health Engineering Abstracts; SIW—Sewage and Industrial Wastes; WPA—Water Pollution Abstracts (Great Britain).*

AQUATIC ORGANISMS

A Study in the Chemical Control of Aquatic Vegetation. M. M. Boschetti. Sanitalk, **5**:21 ('57). In 1953, program of aquatic weed control was initiated by Mass. Depts. of Public Works, Public Health, and Natural Resources. Aim is to discover chems., which, as well as having herbicidal properties. will be safe, economical and easy to handle. Field trials show that sodium arsenite, which is relatively harmless in dild. form, is promising in control of submerged aquatic weeds. Another chem. which shows favorable results is algicide 2,3-dichloro-1,4-naphthoquinone (Phygon) but this does not affect root system. In addition to field trials, lab. expts. to study effects of herbicides on various aquatic plants and bioassay studies for evaluation of acute toxicity of certain herbicides to fish and mammals, will be carried out.—*WPA*

Plankton in River Water to be Used as Water Supply. P. Mangerel. Quarterly Bul., Centre Belge Etude et Document. Eaux, **36**:105 ('57). General discussion of plankton problems in river water which will be treated for water supply use with special reference to conditions in France. Charts are presented comparing plankton contents at 11 stations on Marne R. in spring of 1954 and 1956 and autumn 1956. Similar data are given for Seine and Yonne R. using autumn 1953, winter 1955–56, and autumn 1956 as comparison seasons.—*PHEA*

The Enumeration and Collection of Marine Phytoplankton. R. W. Holmes & T. M. Widrig. J. Conseil Intern. Exploration Mer. (Den.), **22**:21 ('56). Statistical methods were used to study precision with which abundance of marine phytoplankton can be detd. in sample by Utermohl sedimentation method. Adequacy of sampling methods to give accurate estimates of phytoplankton content and distr. in relation to areas under investigation was evaluated. Certain concepts emerging from studies are discussed in regard to best way in which to sample and enumerate marine phytoplankton.—*WPA*

The Relation of Photosynthesis by Phytoplankton to Light in Lakes. W. T. Edmondson. Ecology, **37**:161 ('56). In study of productivity of Bare Lake, Alaska, data were evaluated by means of extensive calcns. made using published measurements of rate of photosynthesis by phytoplankton in relation to light intensity and penetration of light into water. Anal. of published data and derivation of generalized computations for use in limnological studies are considered in detail, and examples of application to specific lakes are described.—*WPA*

Some Relations of Faunal Distributions to Salinity in Estuarine Waters. G. Gunter. Ecology, **37**:616 ('56). Study was made from 1951 to 1953 of lower Mermentau R. basin in La. In marsh in lower part of basin there are 2 lakes, Grand Lake and White Lake, through which river runs. Both lakes are weakly saline and not given to rapid changes in salinity. Numbers, and number of species, of fresh-water animals decreased markedly from Grand Lake through White Lake to Little Bay, although differences in avg. salinity were small. Numbers of marine animals did not differ much in 2 lakes, but were considerably greater in bay. In all 3 areas, marine species predominated over fresh-water species in both species and number of specimens. Effect of low salinities on distr. of estuarine animals is discussed.—*WPA*

The Problem of Plant Disappearance in Hydrogen Sulfide: *Oscillatoria* Lakes. E.-M. Bursche. Z. Fisch., N.F. **IV**, No. 1/2, 53 ('55). Type of lake known as hydrogen sulfide: *Oscillatoria* lake occurs frequently

(*Continued on page 66 P&R*)

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(*Continued from page 62 P&R*)

in neighborhood of Berlin and was regarded by earlier observers as subtype of eutrophic lake. These lakes are characterized by mass development of *Oscillatoria redekei* and *O. agardhii*, and by scarcity of submerged plants. Author carried out investigations to test supposition that this scarcity was caused by low penetration of light. 3 lakes of this type and 2 normal eutrophic lakes were studied. Quant. and qual. detns. of plankton were made, surface and submerged plants were examined, light penetration was measured, oxygen, hydrogen sulfide, and acid-combining capac. were measured, and tests were made for presence of iron, nitrogen, sulfate, and phosphate. Penetration of light was found to be generally greater in hydrogen sulfide: *Oscillatoria* lakes than in normal eutrophic lakes and scarcity of plant growth must be ascribed to antagonistic action of *Oscillatoria redekei*.—*WPA*

Importance of Concentration of Plankton and Their Quantitative Determination in Surface Waters. B. DIETERICH & H. STEINECKE. Vom Wasser (Ger.), 22:72 ('55). Discussing importance of plankton in surface waters from point of view of water supply, authors deal with effects of plankton on filtration of water and with plankton as cause of taste and odor. Types and concns. of plankton causing taste and odor are given. Influence of plankton on oxygen content of water may improve its qual. Methods of detn. of concn. of plankton by means of plankton net, sand filters, and membrane filters are then described and illustrated, and statistical anal. of results is described.—*WPA*

CHEMICAL ANALYSIS

Determination of Bromine and Iodine in Natural Waters. T. A. GONCHAROVA. Gidrokhim. Materialy (USSR), 23:138 ('55). Detailed procedure is given for titrimetric detn. of bromine and iodine in water. Concns. as low as 1 μg can be detd. with error of 7%.—*WPA*

The Complexometric Determination of Calcium in Natural Waters With the Aid of Murexide as Indicator. N. G. FESENKO. Gidrokhim. Materialy (USSR), 23:158 ('55). Detailed procedure is given for complexometric detn. of calcium with Trilon B, using Murexide as indicator. Interference is caused by copper, manganese, and zinc, and by iron in concns. greater than 15 mg/l. Synthesis of Murexide from uric acid is described.—*WPA*

Measurement of the Content of C-14 in Hard Ground Water. K. O. MUNNICH. Naturwissenschaften (Ger.), 44:32 ('57). Observations of unexpectedly high content of carbon-14 in lime of fossil bones led to study of content of carbon-14 in ground waters. Results are given for ground water from various pumping plants in neighborhood of Heidelberg. Content of carbon-14 was again unexpectedly high, showing that carbon of biogenic origin was higher in proportion to that originating in limestone than had been assumed. Exchange processes by which biogenic respiration carbon dioxide enters ground water and possibility of some carbon-14 coming from recently deposited limestone are discussed.—*WPA*

Chemical Analysis by Ultraviolet Filter Photometer. II. Determination of Small Amounts of Chloride. T. KATO & K. SHINRA. J. Chem. Soc. Japan, 77:245 ('56). Concn. of chloride ions of less than 100 μg/ml of water will react with silver chromate, liberating equiv. amt. of chromate ions which can be detd. by ultraviolet filter photometer at 405 mμ. Sample of 50–70 ml water is treated with 5 ml M ammonium acetate and 300 mg silver chromate and made up to 100 ml. Soln. is filtered and 0.25 ml 4 N aq. ammonia soln. is added to portion before measuring extinction. Optimum pH value for reaction is 5.3–7.5, and for extinction measurement 7.6–10.8. Presence of aluminium, iron, bromide, iodide, sulfide, thiocyanide, lead, and barium, will interfere with this method.—*WPA*

Absorptiometric Determination of Low Concentrations of Chlorides. J. S. SWAIN. Chem. & Ind., p. 418 ('56). Available methods for detn. of low concns. of chlorides are evaluated as to their suitability for rapid and accurate anal. of chloride content of boiler waters and boiler-feed waters. Most promising method was found to be that of IWASAKI, UTSUMI, and OZAWA, using ferric alum and alc. soln. of mercuric thiocyanate and detg. ferric thiocyanate color formed by means of absorptiometer. Investigation of

(*Continued on page 68 P&R*)

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(Continued from page 66 P&R)

method to det. optimum ratio of reagents and sample, influence of pH value, temp. and time, and effect of interfering substances is described, and procedure adopted for anal. of chlorides in concns. of 1–10 ppm and 1–100 ppm is given. Results of typical anals. are given in table, showing accuracy of method to be within ±1.0 ppm.—*WPA*

Iodimetric Dead-Stop Determination of Chlorine, and the Colorimetry of Chlorine in Water Chemistry. J. Holluta & H. Meissner. Z. Anal. Chem. (Ger.), 152: 112 ('56). Procedure for detn. of chlorine concns. as low as 0.3 mg/l, with accuracy within ±1%, is given. Blank of app. is detd. by immersing electrodes in very dil. sulfuric acid. Potential of 200 mv is applied and soln. is stirred for 30 min, during which time medium is changed several times. Cell is then refilled with 200 ml of sample, 20 ml of 0.1 N sulfuric acid and 10 ml of 2% potassium iodide soln. Stable zero value on galvanometer is essential. 2 ml of previously standardized 0.001 N sodium thiosulfate are added and mixt. is titrated with 0.001 N potassium iodate. Usually 3 titrations are necessary for attaining a stable endpoint. Solns. standardized by dead-stop detn. were used to assess accuracy of 2 colorimetric methods for detg. chlorine, using *o*-tolidine and *p*-phenylenediamine respectively. Use of *p*-phenylenediamine in the usual manner gives low results with concn. of 0.3 mg chlorine/l and error increases with increasing concn. of chlorine. Use of *o*-tolidine at pH 0.3–1.7, and of potassium fluoride, permits detn. of 0.25–1 mg chlorine/l in presence of 10 mg iron/l with accuracy within ±1%. 1 ml of 5% potassium fluoride is added to 70 ml of sample which is then added to soln. of *o*-tolidine at 20°C; 1–2 ml of concd. hydrochloric acid is then added, mixed and color is detd. after 1 min.—*WPA*

Catalytic Detection of Cyanide Through the Benzoin Condensation. F. Feigl & A. Caldas. Mikrochim. Acta., p. 992 ('55). Authors describe method to detect cyanide through benzoin condensation. Test soln. is

(Continued on page 70 P&R)

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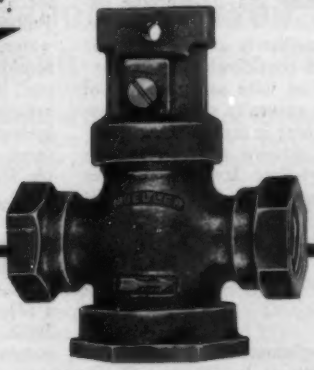

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(*Continued from page 68 P&R*)

treated with alk. benzaldehyde and mixt. is heated for 2–5 min on steam bath. 1 drop of *o*-dinitrobenzene in 5% benzene is added and heating is continued. In presence of cyanide, intense violet color develops in 0.5–3 min. About 0.01 μg of cyanide can be detected at concn. of 1 in 5×10^8.—*WPA*

Detection of Cyanide Compounds in Waters. K. Knie & H. Gams. Ost. Wasserw., **8**:282 ('56). Detection of comparatively high concns. of simple or unstable complex cyanides by titration with silver nitrate and preliminary treatment of solns. containing complex cyanides or in which chlorides are present, are described. Author then deals with detection of small quant. of cyanide and gives details of method described by Stumm, Woker, and Fischer.—*WPA*

Determination of Fluorine in Water Supply. H. Benger. Zbl. Bakt., **163**:312 ('57). Report of paper presented at meeting of the Osterreichische Gesellschaft fur Mikrobiologie und Hygiene in Sep. 1956. Usual methods for detn. of small amts. of fluorine in water, depending on color reaction with zirconium oxychloride and alizarin sulfonic acid, are not suitable for routine investigations because of slow color development, strong dependence on time, and disturbance by interfering substances. Reaction with zirconium oxychloride and Eriochrome cyanine in hydrochloric acid soln. is preferable. Color develops immediately and is stable for at least 24 hr, and considerably higher extinctions are obtained with same concns. of fluoride. In amts. generally present, sulfate is only interfering ion; it can be removed by pptn. with barium chloride or its effect can be calcd. Lambert-Beer law applies over range 0–1.4 mg fluorine/l. Method is suitable for natural waters without previous distr. Spectrum photometer with sufficiently small transmission width is necessary; reaction is not suitable for visual colorimetry. —*WPA*

A Simplified Complexometric Method for Determining Hardness in Water. B. V. Donskoi. Energetik (USSR), **4**:25 ('56). Mixed indicator for use in detn. of hardness consists of 2 g of ammonium chloride, 10 cc of 25% ammonium hydroxide, 0.4 g of nitroxyl sulfide, and 4 cc of indicator soln. dild. to 100 cc with water. Indicator is stable for 10–12 hr, and 2.5–3.5 cc are added to 100-cc sample. Color may be estd. visually, or soln. titrated with sodium ethylenediamine tetraacetate.—*WPA*

A Spectrophotometric Determination of Iodine in Natural Waters. K. Sugawara; T. Kayama; & K. Terada. Bul. Chem. Soc., Japan, **28**:494 ('55). Method is described for spectrophotometric detn. of iodine in natural waters. Iodine is first pptd. with silver nitrate soln., large amt. of silver chloride acting as coprecipitant. It is then oxidized to iodate with bromine. Iodate is treated with cadmium iodide, when 6 times amt. of iodine contained in original sample is liberated. This is detd. spectrophotometrically as starch-iodine complex.—*WPA*

A Sensitive and Rapid Method for the Determination of Iron in Natural Waters. G. Almassy & M. Z. Kaval. Magyar Chem. Folyoirat (Hung.), **62**:325 ('56). In method for detn. of iron in natural waters, 0.5 ml of N nitric acid is added to 10-ml sample which is kept at 100°C for 5–10 min. Then 0.5 ml of aniline reagent and 1 drop of bromophenol blue indicator are added, and pH value is adjusted to about 4.7 with 15% sodium acetate; 0.2 ml of dipyridyl reagent is added and soln. is kept in hot-water bath for 5 min. Light extinction is measured in Pulfrich photometer against blank of distd. water, and concn. of iron is detd. from calibration curve. If original water contains ppt., it must be be shaken well before 50-ml sample is taken. To this 0.5 ml of 20% hydrochloric acid is added, and soln. is filtered. Filtrate is then used as original sample for method described above. Sensitivity is 30 μg/l.—*WPA*

Determination of Iron in Tube Well (Underground) Water. S. N. Mitra & P. B. Sarkar. Current Sci. (Ind.), **26**:111 ('57). Water from some tube wells in India is clear when freshly drawn, but becomes opalescent on standing as result of pptn. of ferric hydroxide. Iron so deposited usually adheres to sides of bottle and is not dislodged even when water is well shaken before anal. Thus detn. of iron in water only does not give accurate est. of total concn. of iron in well water. Authors have

(*Continued on page 72 P&R*)

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(*Continued from page 70 P&R*)

developed method to overcome this difficulty. Vol. of sample is measured, and concn. of iron in well shaken water is detd. by colorimetric thiocyanate method. Bottle is then emptied, rinsed gently with little distd. water, and about 5 ml of hot dil. hydrochloric acid are poured into bottle to dissolve iron deposit. Soln. is then poured into beaker, bottle is washed out with distd. water which is added to acid soln. Soln. is boiled with concd. nitric acid, and concn. of iron is detd. colorimetrically. Iron in deposit is calcd. as parts/100,000 of original sample, and result is added to previously detd. concn. of iron in water. Results obtained with this method agree well with concn. of iron detd. in freshly-drawn well water in which no deposition has occurred.—*WPA*

Determination of Magnesium with Titan Yellow. A. BUSSMAN. Z. Anal. Chem. (Ger.), 148:413 ('56). Titan yellow in alk. soln. (pH 12) is yellow-brown in color and gives with colloidally dissolved magnesium hydroxide red adsorption compd. which can be measured photometrically. Amt. of Titan yellow varies with range of measurement; for 0–50 μg of magnesium in 50 ml, 5 ml of 0.01% aq. soln. of Titan yellow is sufficient. Soln. of reagent is stable if kept in dark at 0°C. Alk. of sample is adjusted with 10 ml of 2 N sodium hydroxide. Interfering ions and their removal are described.—*WPA*

The Quantitative Analysis of the Alkaline-Earth Metals by Paper Chromatography. F. H. POLLARD; J. F. W. MCOMIE; & J. V. MARTIN. Analyst (Br.), 81:353 ('56). Paper-chromatographic procedure is described for quant. anal. of low concns. of mixt. of barium, strontium, calcium, and magnesium. Metals are separated as formates by downward elution on twin-strip chromatogram of acid-washed filter paper, with solvent consisting of methanol, *iso*-propanol, formic acid, water, and ammonium formate. They are extracted and detd. spectrophotometrically, with *o*-cresolphthalein complexone for barium, strontium, and calcium, and with Eriochrome black T for

(*Continued on page 74 P&R*)

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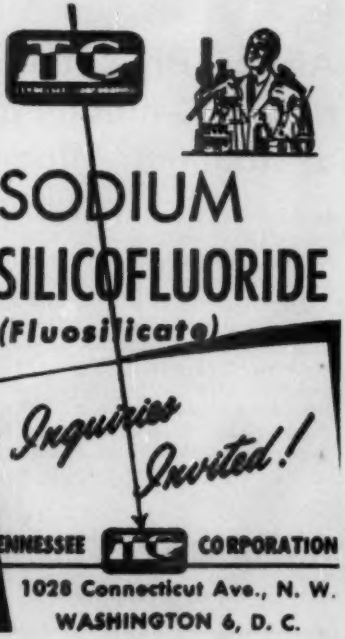

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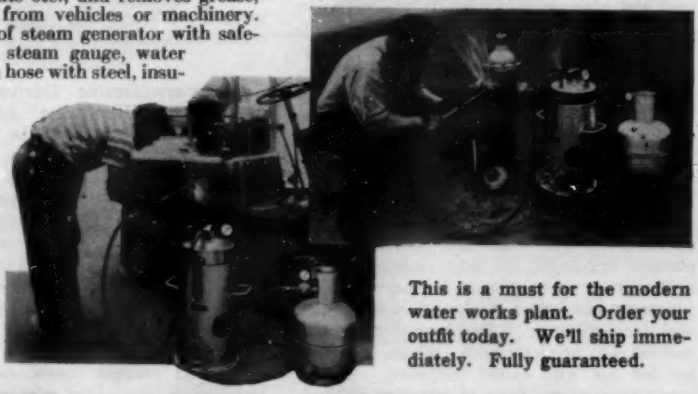

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(*Continued from page 72 P&R*)

magnesium. Method is particularly suitable for detection of small amts. of magnesium, which may be detd. in presence of 500-fold excess of calcium, with error of less than ±2%.—*WPA*

The Spectrophotometric Determination of the Alkaline-Earth Metals With Murexide, Eriochrome black T, and with *o*-cresolphthalein Complexone. F. H. POLLARD & J. V. MARTIN. Analyst (Br.), 81:348 ('56). Use of absorptiometric methods for detn. of low concns. of barium, strontium, calcium, and magnesium after their sepn. by paper chromatography, is discussed. Modified procedures are given for detn. of calcium with Murexide and of magnesium with Eriochrome black T, and new methods are described for detn. of strontium with Murexide and of barium, strontium, and calcium with *o*-cresolphthalein complexone.—*WPA*

Determination of the Nitrate Ion in Water. R. MULLER & O. WIDEMANN. Vom Wasser (Ger.), 22:247 ('55). Authors describe investigation into accuracy and conditions of application, in detn. of nitrate, of oxidation methods (diphenylamine-sulfuric acid and brucine-sulfuric acid) and nitration methods (sodium salicylate and phenol disulfonic acid). Taken into account for judgment of 4 methods were detn. of absorption sensitivity for different wave lengths, detn. of stability of color formation, detn. of color intensity with increasing concn., and suitability for purely visual examn. 2 oxidation methods were unsuitable for use with photometer; variability of reagent solns. and of reaction colors made preparation of standard curve or of type solns. with constant color content impossible. Diphenylamine-sulfuric acid method is unsuitable for purely visual judgment and better results are not obtained with colorimeter. Brucine-sulfuric acid method was accurate with Dubosc colorimeter but not with visual judgment. Results never reached precision of nitration methodods. 2 nitration methods are regarded as ideal for operation with optical app. With clear water, single standard curve is accurate to ±0.25–±0.5 mg/l, and results are reproducible. From point of view of sensitivity to interfering factors, sodium salicylate method is preferred; in suitable concns. purely visual determination gives accuracy of ±0.8 mg/l.—*WPA*

Determination of Nitrates in Aqueous Solution. J. UNGAR. J. Appl. Chem. (Lond.), 6:245 ('56). Volumetric method for detn. of nitrate is described, utilizing decolorization of soln. of indigo carmine by dil. nitric acid in presence of strong sulfuric acid. Titrating soln. also acts as indicator. Some interferences and their circumvention are discussed, including that of nitrite. List of references is appended.—*WPA*

The Determination of Nitrate in the Presence of Nitrite in Dilute Aqueous Solution. E. G. WHITE & K. A. MURRAY. J. S. African Chem. Inst., 9:66 ('56). It is usually considered that the most suitable rapid routine method for detg. nitrate in aq. solns. such as sewage effluent is colorimetric procedure using phenol disulfonic acid; nitrite interferes in this method but cannot be readily destroyed without loss of nitrate. Electrolytic oxidation method using low current densities has been found to be satisfactory for oxidation of nitrite and this forms basis of new method in which nitrite is detd. by Griess-Ilosvay method and separate portion of nitrite is oxidized electrolytically to nitrate which is detd. by phenoldisulfonic acid method. Difference between 2 values gives concn. of nitrate.—*WPA*

***p*-Phenylenediamine Derivatives as Reagents for Ultraviolet Absorptiometric Determination of Nitrite Ion.** D. F. KUEMMEL & M. G. MELLON. Anal. Chem., 28:1674 ('56). Report is given of investigation made into use of diazonium salts of aromatic amines for ultraviolet absorptiometric detn. of nitrite ion. Of amines used *p*-phenylenediamine derivatives were shown to be most sensitive; absorption maxima occurred in 330–380 mμ region. Effect of nitrous acid on absorption spectra of *p*-phenylenediamine derivatives, effect of pH value and reagent concn. on spectra, and stability of diazonium salts and of reagents were examined, and mole-ratio studies were made. Chloro-*p*-phenylenediamine system was investigated in greater detail for reproducibility, reagent stability, and interference effects. Results are discussed, and advantages of method over other colorimetric methods for detn. of nitrite ion, involving separate diazonium coupling step, are given. —*WPA*

(*Continued on page 76 P&R*)

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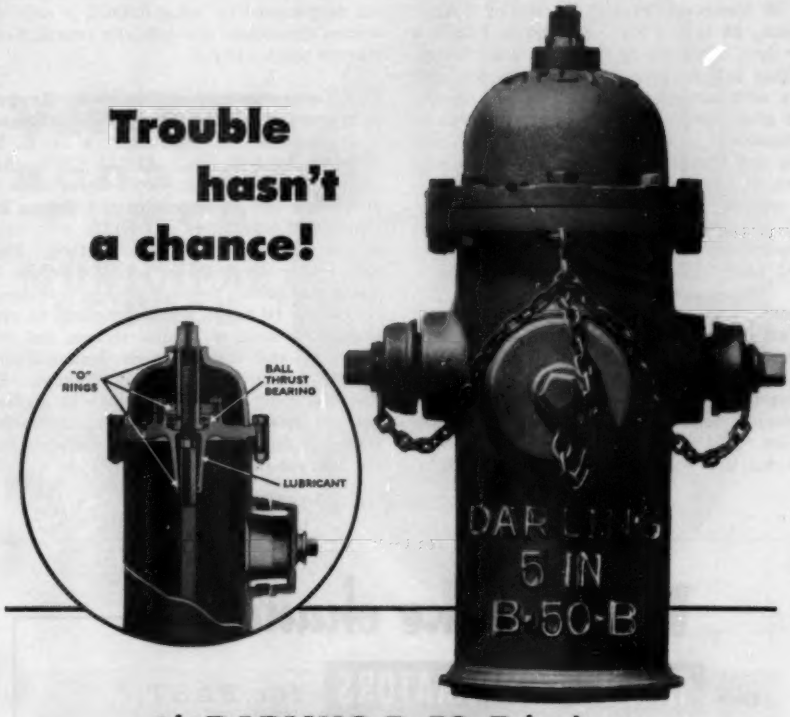

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(*Continued from page 74 P&R*)

Rapid Determination of Nitrate Nitrogen in the Presence of Ammonia and Urea. R. M. ENGELBRECHT & F. A. McCOY. Anal. Chem., **28**:1619 ('56). Method is described for detn. of nitrate by reduction with ferrous sulfate and titration of excess ferrous sulfate with potassium permanganate. Results are given for detn. of nitrate in presence of ammonium, potassium, sodium, chloride, sulfate and phosphate ions, and in presence of urea, in which case sodium chloride is added to sample. Method is especially suited for est. of nitrate in mixed fertilizers, and is improvement over conventional methods in both anal. time and accuracy.—*WPA*

Determination of Dissolved Oxygen in Water. R. FREIER & G. RESCH. Z. Anal. Chem. (Ger.), **148**:427 ('56). By suitable modification, Winkler method for detn. of oxygen in water in presence of interfering reducing substances can be used for amts. down to 20 μg oxygen/l. Titration can be carried out without oxidation of iodide if pH value of 2.3–2.5 is maintained by addn. of acetate buffer soln. Sensitivity mentioned can be obtained by using 0.00625 *N* soln. of sodium thiosulfate and specially standardized reagent solns.—*WPA*

The Determination of Dissolved Oxygen in Water Containing Reducing Substances. I. DAVIES; M. N. REDFEARN; & D. E. Y. REMER. Analyst (Br.), **81**:113 ('56). Account is given of exptl. work undertaken to investigate use of ion-exchange columns for removal of substances interfering with detn. of D.O. in water by Winkler method. Procedure was developed in which water to be tested was passed through column containing Amberlite IRA-400 resin converted to permanganate form to oxidize ferrous and sulfite ions, and then through demineralizing column before detn. of D.O. content. By this treatment Winkler test may be applied, without modification, to waters containing traces of reducing substances; sulfide interference was not eliminated.—*WPA*

(*Continued on page 78 P&R*)

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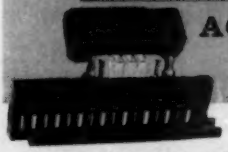

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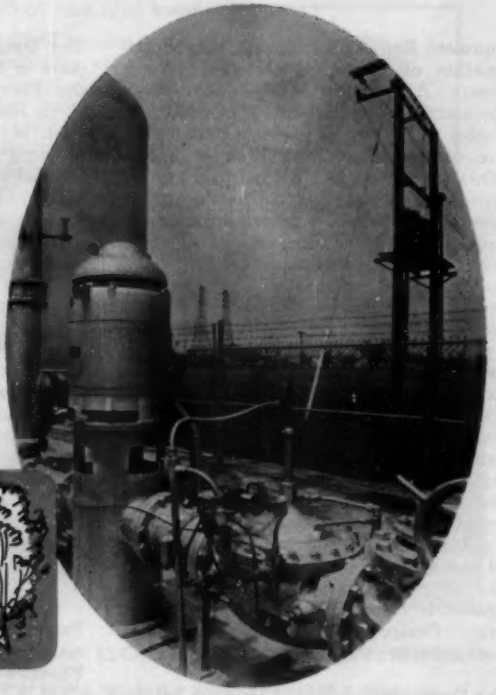

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(*Continued from page 76 P&R*)

Improved Rapid Colorimetric Microdetermination of Dissolved Oxygen. W. F. Loomis. Anal. Chem., 28:1347 ('56). Rapid colorimetric method for detg. D.O. in 0.5-ml samples of water, as previously described, gave, on comparison with Winkler method, calibration curve which did not pass through origin and was slightly concave. 3 modifications, and other improvements, are reported, which have been devised to obtain strictly linear calibration curve passing through origin and to enable method to be used as an alternative to standard Winkler method.—*WPA*

Continuous Measurement of Dissolved Oxygen in Water. H. S. Levine; W. V. Warren; E. C. Tsivoglou; & W. W. Walker. Anal. Chem., 28:343 ('56). Method has been developed for continuous measurement of D.O. in water, based on establishment of Henry's law equil. between flowing water sample and concn. of oxygen in gas at constant pressure and vol. Results are not affected by temp., salt concn., presence of polg. material, aeration constant, barometric pressure or concn. of D.O. in water. Design and performance of practical instrument for detn. are described.—*WPA*

The Determination of the Oxygen Saturation Concentration. H. Schmassmann. Schweiz. Z. Hydrol. (Switz.), 18: 144 ('56). Oxygen satn. concn. in water differs in equil. with dry air and with humid air. This fact has not usually been considered in establishing concn. values in limnology. It has usually been assumed that air in immediate contact with water is satd. with water vapor, but this is actually rarely so. Humidity of this layer of air varies greatly from time to time and from place to place, and therefore detns. of oxygen satn. values have no real significance in field work unless this actual humidity is known. Author concludes that it is sufficiently accurate in field work to use oxygen satn. values from air with 100% humidity. In addn. changes in air pressure alter oxygen satn. values of water. In field work, it is sufficient to use avg. air pressure for given altitude rather than abs. pressure at moment of sampling. Equation is given for detg. value of oxygen satn. concn. at given water temp. and given altitude. Usefulness of tables and nomograms of oxygen satn. values is discussed.—*WPA*

The Determination of Potassium and Sodium in Drinking Water With the Aid of the Flame Photometer. E. L. Molt & T. L. Hien. Chem. Weekbl., 52:313 ('56). Amt. of K and Na in drinking water is assuming increasing importance because of radioactivity of natural K and increased medical importance of these 2 elements. Authors studied influence of Ca and Na on results obtained for K, and of Ca and K on those obtained for Na. For detn. of K, interference from Ca and Na varies when these are present in low concn., but reaches a constant value for amounts of Na > 100 mg and of Ca > 50 mg/l. For detn. of K, add 10 ml of soln. containing 0.3% of Ca^{2+} and 0.2% of Na^{+} as chlorides to 100 ml of a soln. containing 1% of K^{+} as KCl to calibrate instrument; add same amt. when making detn. Accurate and reproducible results for K^{+} are claimed with this procedure. In detn. of Na^{+}, there is interference from K^{+}, but not from Ca^{2+}, and 100 mg of K^{+}/l of sample should be added. Anions do not affect results of flame-photometric detn. of K^{+} or Na^{+}.—*PHEA*

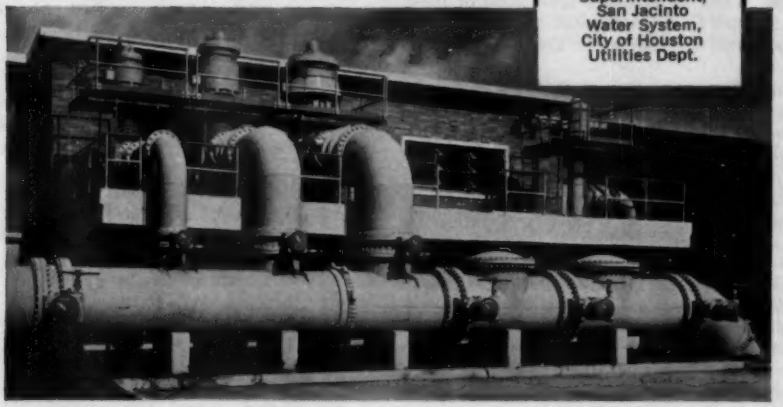
Paper-Chromatographic and Electrophoretic Investigations on Simple and Higher Phosphates. W. Frank. Vom Wasser (Ger.), 23:30 ('56). Author deals with constitution of simple and complex phosphates and describes method of paper-chromatographic sepn. and detn. of simple and complex phosphates and the sepn. of phosphates by paper-electrophoresis. Quant. detn. of separated phosphates is also described. By paper-chromatographic method it is possible to separate phosphates of different degrees of condensation and of different constitution. Examples are described of results of use of these methods of sepn. and anal.—*WPA*

Determination of Strontium-90 Activity in Water by Ion-Exchange Concentration. C. W. Stanley & P. Kruger. Nucleonics, 14:114 ('56). Authors describe methods for detn. of strontium-90, from radioactive fallout in natural water and in nuclear reactor waste waters. Preliminary concn. is necessary as large vols. of water have to be handled to isolate small quant. of strontium. Evapon. filtration of ppts. and ion-exchange methods are used. For large vols. of water, ion-exchange method is most economic. Method is outlined. Expts. are described to

(*Continued on page 80 P&R*)

The Houston Water Purification Plant was one of the first plants to use rubber-seated butterfly valves in filter service. Freese, Nichols and Turner, Consulting Engineers, chose R-S Rubber-Seated Butterfly Valves over conventional gate valves for all low-pressure service because their compact design permitted close coupling of pipe. Result: a substantial saving in space, and a direct, in-place cash saving of $124,000.

CLYDE R. HARVILL, Superintendent, San Jacinto Water System, City of Houston Utilities Dept.



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When the Houston Purification Plant went into operation in 1954, 65 R-S Rubber-Seated Butterfly Valves were used to equip the entire low pressure system. Today, Mr. Harvill reports that these valves are giving good service, and that he is fully satisfied with their performance and minimum maintenance requirements during their first years of operation.

R-S Rubber-Seated Butterfly Valves are easy to operate, and will provide 100% effective shut-off with a 125-pound pressure drop. Wedging action of the vane against the tough, flexible rubber seat assures positive closure, even around shaft bosses. The special composition of rubber and other materials gives long wear and resistance to the abrasive action of material flow.

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(*Continued from page 78 P&R*)

det. applicability and ease of ion-exchange concn. of strontium-90 from large vols. of water, which include elution curve to test conditions for selective elution of yttrium from strontium; test runs to det. recovery of strontium from water passing through ion-exchange column using conditions of former expt. Data to illustrate application of these techniques to det. strontium concn. in Pittsburgh rain and tap water are given.—*WPA*

Rapid Determination of Sulfates in Drinking Water by Spectrophotometric Methods. 1. The Aging of Barium Sulfate Solutions. M. Brogioni. Chimica (Milan), 12:423 ('56). During some expts. to det. sulfate ion concn. in drinking water by measuring spectrophotometrically opacity induced in solns. by addn. of barium chloride soln., it was found that results differed according to age of latter soln. It is suggested that molecules of barium chloride in soln. gradually increase in size by aggregation phenomena, and large molecules act as crystallization nuclei when they come in contact with sulfate ions.—*WPA*

Volumetric Determination of Sulfate Ion in Water. M. S. I. Acuna. Rev. Obras Sanit. Nacion (Buenos Aires), 20:30 ('56). Method is given for volumetric detn. of sulfate ions in water. Excess of 0.02 N barium chloride soln. is added to water sample, and approx. titration is made to Eriochrome black T endpoint with 0.02 N Versene. 2 ml of 0.02 N magnesium chloride are then added and a final accurate titration is made with addnl. 0.02 N Versene. Results by this method were found to be as accurate as gravimetric method for concns. between 10 and 250 mg sulfate/l. Results, given in tables, show figures obtained by both methods for various water samples. Copper interferes with endpoint if present in amts. greater than 0.2 mg/l. Amts. of up to 10 mg/l of ferric iron do not interfere with endpoint.—*WPA*

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How to Store Large Tonnages of Rock Salt in Limited Space

One of the best ways to increase salt-storage capacity, without taking up valuable space near your water-softening installation, is to use a "Silo Lixator." Developed by International, this space-saving unit actually *combines* salt storage with brine making—and maintains a ready supply of fully saturated brine for regenerating zeolite and/or base-exchange resins.

The Silo Lixator makes excellent use of limited storage space, because it stores rock salt in a high, vertical column. The upper portion of the column stores dry salt. The bottom of the column rests in a dissolving tank, where water is added to make brine. Whenever salt in this tank is dissolved, it is immediately replaced by dry salt flowing down from the upper portion of the column.

For storing any given amount of rock salt in dry form, ample space can be provided in the Silo portion, by building it to the proper height. The rugged steel, wood-stave, or prefabricated-concrete construction of the Silo assures long, dependable service.

Rehandling eliminated. Because dry salt stored in the Silo Lixator is dissolved to make brine, there is never any salt rehandling in the plant. A simple piping system, installed where it won't interfere with other operations, carries this brine to points of use. What's more, every drop of Lixate Brine is completely self-filtered by the rock salt crystals at the bottom of the dissolving tank. So you always have clear, 100%-saturated brine—the brine that always gives you complete, economical zeolite regeneration.

Incoming salt shipments need no special handling. One of the more common delivery methods works like this: rock salt flows directly from a hopper car into a screw-conveyor trough, which, in turn, carries the salt to a bucket elevator. Another unloading method, possible when the elevator boot is low enough to permit gravity feed from the hopper car, is the gravity-feed chute.

Another money-saving feature: The Silo Lixator needs to be cleaned out only once every 12 to 18 months. The clean-out process is very simple: a valve or door is opened at the bottom of the dissolving compartment. All accumulated impurities are flushed through the clean-out, and the Silo Lixator is ready for use . . . needs no further attention for at least another year.

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INTERNATIONAL SALT CO., INC.
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(*Continued from page 50 P&R*)

then getting back into the tank, each break consumed more than an hour. City officials rebelled, the repair crew was adamant, and only the intercession of AWWA member Ray Brax, a cafe owner as well as Assaria water superintendent, broke the deadlock with the offer to furnish the men with coffee, inside the tank, twice daily—which he accomplished by climbing the ladder and then lowering the pot and cups by rope and bucket.

What we'd prefer, though, on some of these really hectic days is a martini break or a siesta, or, better yet, both—that is, if they could be worked in so as not to interfere with the coffee breaks. The only real problem, actually, is that we'll soon have so many breaks we won't be able to get them all in when we go to the 4-day week.

A larger town of Bethlehem was in the news last Christmas with a story of good will toward men. Actually, it was the Bureau of Water of adjoining Allentown, Pa., that displayed the Christmas spirit in attaching spigots to its fire hydrants along the Bethlehem border line to permit Bethlehemites to draw all the water they needed until their own supply was palatable again. Potable the Bethlehem water was, but it was muddy because of construction work north of its Wild Creek watershed and reservoir in the Pocono mountains—so muddy that residents had resorted to bottled water and rural springs before Allentown came to the rescue. All Bethlehem's water has to do is lie still a while, of course.

Long time no sea still seems true as far as public water supply is concerned, although the price of desalting sea water has been yielding to the extensive research now being conducted all over the world. A good measure of the yield was obtained last November in Washington at the eleven-nation Symposium on Saline Water Conversion conducted by the Dept. of Interior's Office of Saline Water. There, figures from $0.10 to $1.25 per 1,000 gal were being bandied about by the 200 experts in attendance. The dime per thousand was an undetailed claim by one Alexander Zarchin for a freezing process that he is working on for the Israeli government. Here in the US, the lowest figure mentioned was by the Maxim Silencer Co., which claimed to be able to desalt sea water for 20 cents per thousand by a steam-jet vapor compression system, but it appeared that this figure was based on obtaining cost-free energy and taking credit for auxiliary use of heat from steam generated in the process. The official estimates on government-aided projects were $0.39 per 1,000 gal for multiple-effect distillation, $0.65–$1.15 for flash distillation, $0.96 for rotary vapor compression distillation, and $0.75–$1.25 for solar evaporation. All these figures, of course, are predistribution figures, although they are most often compared with present costs of water delivered.

Among the practical systems discussed, a highlight was one expected to go into service in April to serve 3.6 mgd to a number of gold-mining companies in Orange Free State, South Africa, reducing the salt content of a brackish water from 4,000 to 500 ppm at approximately 29 cents per 1,000 gal. But a lot more stir was caused by a theoretical system described by erstwhile JOURNAL author Gilbert Levin, now of Resources Research, Inc. The

(*Continued on page 84 P&R*)

New Salem pipe line to augment present system will add approximately 50 MGD capacity to city water supply system.

a line of growth... for Oregon's capital city

American Concrete Cylinder Pipe is helping Salem, Oregon to grow

A simple but important fact is that no city, however great its potential, can grow beyond the capacity and dependability of its main water supply system.

Recognizing this fact, and envisioning a population in 30 years that will require three times the maximum capacity of present water transmission lines, Salem, the capital city of Oregon, is carrying out a program designed to meet future needs.

Under the direction of the City's Water Department Manager, John L. Geren, and the technical supervision of Consulting Engineers Clark and Groff of Salem, an 18-mile pipe line of 48" and 54" diameter American Concrete Cylinder Pipe is being installed by Lord Brothers, a general contracting firm of Portland, Oregon. This new line will triple the present capacity of the supply system.

The performance record of this type of reinforced concrete cylinder pipe has been so outstanding throughout the West that Salem can be confident that this water "growth line" will be giving efficient, economical service for many, many years to come.

Strength, permanence, sustained high carrying capacity, and trouble-free service are characteristics of American Concrete Cylinder Pipe which make it the right pipe for this forward looking city.

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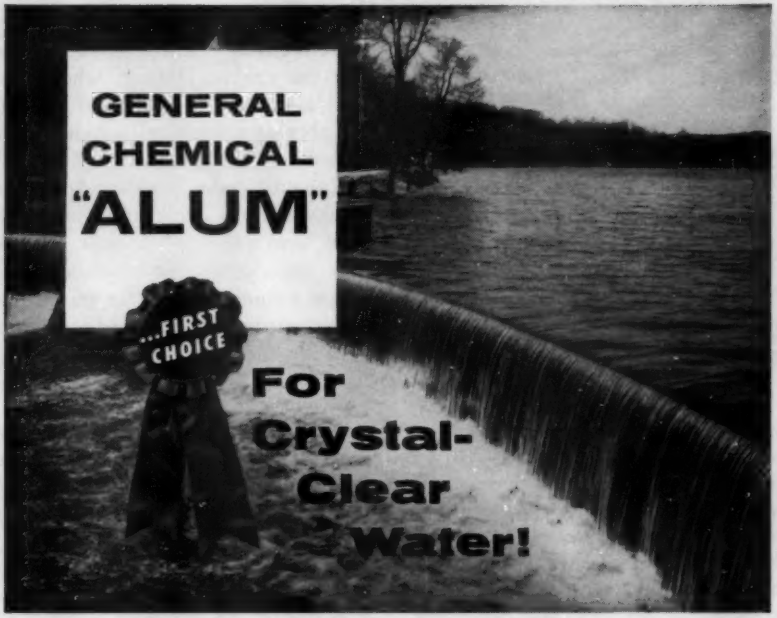
(*Continued from page 82 P&R*)

Levin system involves the use of reversible salt-accumulating algae which would remove salt from one basin (in which conditions would be set to promote maximum pickup) and then drop it into another basin where conditions would be reversed, very much as ion-exchange resins pick up hardness ions and then release them on recharge. By circulating his algae through the two types of basins, he indicates that it would be possible not only to desalt the water, but, from the excess of algae obtained through the reproduction of the workers, to recover a bonus of protein. Although no one has yet agreed with Gil that the system can be worked, he points out that no one has

US Army photo

In a ceremony at Brooke Army Medical Center, Fort Sam Houston, Tex., certificates of competency are being awarded to the first military personnel to qualify as Class C water works operators in Texas. Brigadier General Elbert DeCoursey (left), commandant of the Army Medical Service School, is shown presenting certificates to M/Sgt. Wyllis E. Cauvel, M/Sgt. Clifton B. Sawyer, Sgt. 1/c Wallace W. Wright, Sp. 2/c Eugene A. Jankowski, and Cpl. Robert A. Gossen, all graduates of a short course held at Trinity University under the auspices of the Texas Water & Sewage Works Assn., in cooperation with Texas A&M and the state health department.

(*Continued on page 86 P&R*)



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Basic Chemicals for American Industry

(*Continued from page 84 P&R*)

disagreed either—and where there's a Gil. . . .

Still *long time no sea!*

William H. Cochrane has been elected president of Neptune Meter Co. He succeeds Dante E. Broggi, who resigned in order to devote more time to outside interests but will continue to serve as a consultant to Neptune. Mr. Cochrane was formerly executive vice-president responsible for coordinating policies and operations among Neptune and its subsidiary companies.

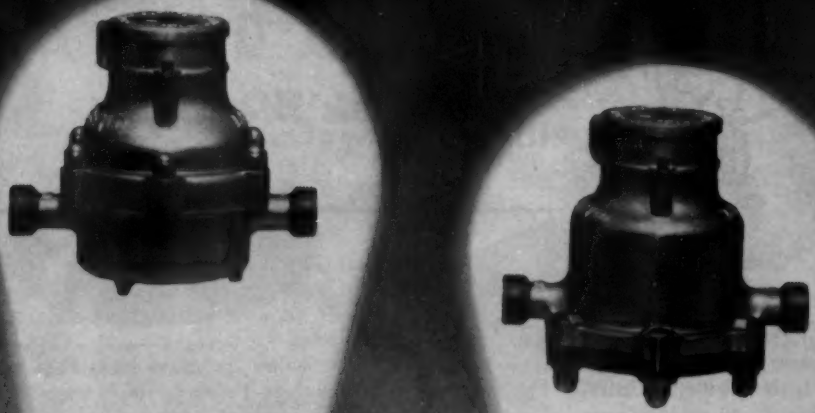
The plethora of poolishness discussed last month as a newly significant factor in public water supply planning (P&R p. 46) has turned out to be even more insidious than we first supposed. Actually the inside story, which we then reported, has proved to be strictly for peasants. The real cool pool is a basement one—presumably in addition to the outdoor plunge. And on Long Island, builder Hillard Man is providing 12 × 18-ft basement *bains* for a mere $800 extra in his Eden Roc ($19,500–$25,200) homes. Deep enough for diving in the center and shallow enough for wading on the side, these subsurface swimmeries ought, actually, to be more of a prize than a problem to water workers, in balancing those peak-loaders of summer backyards. In the water supply field, after all, there's no business like slow business.

Benvenuto Cellini loves water! Not the original Benny, who died in 1571 and probably preferred wine, but his present namesake—a tiger tomcat owned by Mme. Jacqueline Marcault of Cleveland Heights, Ohio—who not only drinks the stuff, but swims in it, washes in it, and plays in it most uncattily. Just unnaturally hydrophilic, Benny makes a running jump for running water whenever or wherever he hears it—in the kitchen sink, in the washbasin, in the tub, or—well anywhere. Not only does he have to be watched on that account, but because he is a constant sud-swiper and bubble-buster when his mistress washes clothes or dishes or, if she forgets to shut the door, herself.

Madame owns no goldfish!

Speaking of quaffing quadrupeds, though, Jewel's Reward and Misty Flight are two more that love water—more specifically, Mountain Valley Water. Ever since they started starring in the sport of kings, in fact, both Jewel's Reward, champion of the Maine Chance Farm, and Misty Flight, favorite of the E. R. Bradley string, have been drinking Mountain Valley Water at a cost of approximately $10 each per day. Having noted that no less a trainer than Sunny Jim Fitzimmons is responsible for Misty Flight's discriminating draught and that he formerly watered such famous horses as Nashua and Bold Ruler similarly, we can hardly argue against the effectiveness of the brew. As far as explaining it is concerned, we can only suppose that its high mineral content makes horses, too, run.

At last reports, Mountain Valley Water was also being served exclusively to the US Senate. Senators, of course, run only once every 6 years.



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Section Meetings

California Section: The 38th annual meeting of the California Section was held in San Jose, Oct. 29–Nov. 1, with 1,225 in attendance, including 560 members and 321 ladies. The headquarters hotel was the St. Claire, and technical sessions and exhibits were held in the Civic Auditorium across the street. The 51 exhibits were well attended.

The program was opened by the Kick-off Luncheon Wednesday afternoon, attended by 252 members, ladies, and guests. General technical sessions were held Wednesday afternoon and all day Thursday. Three parallel division technical programs were held on Friday morning. The meeting concluded on Friday afternoon with a general session, featuring a stimulating and challenging message from President Merryfield, and the business meeting. [A list of technical papers presented appeared on p. 1606 of the December 1957 issue.]

Attendance at the sessions was about as usual, averaging about 200 and reaching a peak of about 300. The business session was attended by 175 members. Social events included the annual golf tournament held on Wednesday morning, participated in by just over a hundred golfers. The All-Division Banquet was attended by 307 members, guests, and their ladies on Wednesday evening. The California Day Luncheon Thursday noon, attended by 197 persons, was highlighted by Fred Camp's inspiring color slides showing the High Sierra watersheds of the Los Angeles Dept. of Water & Power, San Francisco, and the East Bay Municipal Utility Dist. The meeting closed Friday evening with the Manufacturers' Annual Banquet attended by 856 people. This gala affair included entertainment, and ended with dancing to Ed Husseys' orchestra.

The ladies' program included a tour and lunch on Tuesday (186 in attendance) visiting points of interest in the San Jose area, with luncheon at Rickeys in Palo Alto. On Friday, 251 ladies enjoyed luncheon at the Hawaiian Gardens and a fashion show produced by the Joseph Magnin Co.

On Friday afternoon the main event of the business meeting was the election of new Section officers. These include: Duncan A. Blackburn (chief engineer and general manager, Pasadena Water Dept.) —chairman; John A. Luthin (chief engineer, Suburban Water Systems, Puente) —vice-chairman; and R. E. Dodson Jr. (head, Water Production Div., San Diego Water Dept.)—secretary-treasurer.

Edward A. Reinke, Chief of the Bureau of Sanitary Engineering, State Dept. of Health, Berkeley, received the Elliott Award in recognition of his untiring support of water purveyors in protecting the quality of their water sources, and his current effective effort as chairman of the Section Committee on Water Quality. Announcement was also made of the nomination of Harry Reinhardt, of the California Water & Telephone Co., for the Fuller Award.

HENRY J. ONGERTH,
Secretary-Treasurer

Florida Section: The Florida Section held a very successful meeting at Jacksonville, Nov. 10–13, with the greatest

(*Continued on page 90 P&R*)

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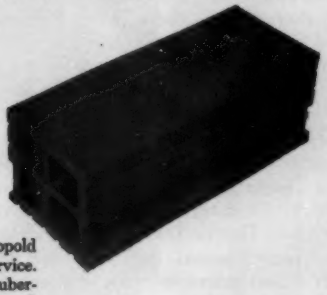

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Section Meetings

(Continued from page 88 P&R)

attendance experienced to date. This was a joint meeting with the Florida Sewage & Industrial Wastes Assn. The total registration was 399. They came early and stayed late—there were 275 registered Sunday afternoon, Nov. 10, and 307 attended the banquet and dance Wednesday night, Nov. 13, which was the last entry on the program.

The social affairs and the technical program were well managed and pleasingly presented by Section Chairman Stanley Sweeney, Local Arrangements Chairman Emory Dawkins, and Program Chairman Thomas deS. Furman. The Club Room activities were efficiently run by Robert L. Burdick, chairman of the Club Room Committee. [A list of the technical papers and their authors appeared on p. 1607 of the December 1957 issue.]

The following new officers and trustees were elected: chairman—Harold D. Overhiser, Mount Dora; vice-chairman—David B. Lee, Jacksonville; secretary-treasurer—John G. Simmons, West Palm Beach; trustees—Charles A. Black, Gainesville, and Leland J. Cobb Jr., Tampa. The Fuller Award nomination went to Sidney W. Wells, of Gainesville, and the A. P. Black Cup for obtaining the largest number of new members during the year was awarded to H. L. Berkstresser of Panama City.

J. D. Roth
Secretary-Treasurer

North Carolina Section: The North Carolina Section held its 37th annual meeting at the Hotel Sir Walter in Raleigh Nov. 11–13, 1957, in conjunction with the North Carolina Sewage & Indus-

(Continued on page 92 P&R)

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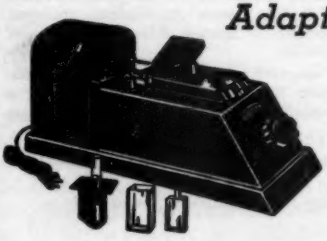

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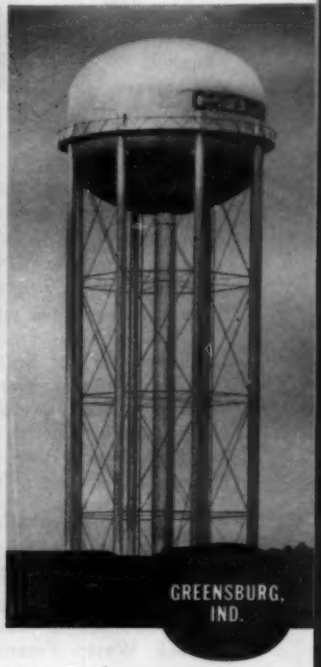

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Section Meetings

(Continued from page 90 P&R)

trial Wastes Assn. One of the highlights of the technical program was a panel discussion on the "Impact of Federal-Aid Highways on Utilities Systems." [A complete list of papers and their authors appeared on p. 1612 of the December 1957 issue.]

At the annual banquet, the Fuller Award nominee was announced as Earle C. Hubbard, State Board of Health, Raleigh. The Maffitt Membership Cup, awarded annually by the city of Wilmington to that member who was instrumental in securing the greatest number of new members for the Section was won again by Mrs. Ethel G. Young of Raleigh. She has permanent possession of the former cup, having won it for the previous 3 years. Lester Rose, the little man who makes big speeches, entertained the members and guests with an address on "It Pays to Be Crazy." Following the banquet, the Association dance was held in the ballroom of the Hotel Sir Walter.

During the meeting an inspection trip was made through Raleigh's new Walnut Creek Waste Treatment Plant. Films were shown on the last morning, followed by the annual business meeting. Officers elected were: W. F. Stokes, chairman; W. E. Long Jr., vice-chairman; D. Y. Brannock, secretary-treasurer; and L. P. Bloxam, trustee.

A most enjoyable aspect of the meeting was the fellowship, fun, and entertainment provided by WSWMA. For the ladies attending the meeting there was a coffee hour and bridge party, luncheon at State College Union, a shopping trip to Cameron Village, and a visit to the governor's mansion.

W. E. Long Jr.
Secretary-Treasurer

Southwest Section: The Southwest Section held its 46th annual meeting in Oklahoma City, Okla., Oct. 13–16, 1957, with Morrison B. Cunningham, Director of Public Works, serving as host. There was a registration of 784, a record for meetings held in the state of Oklahoma.

Three full days were devoted to program papers and discussions, and the program was enthusiastically received by large groups attending each session. The program was well balanced, featuring technical papers on recent advancements in technology, papers designed to interest the small water works operators, and panel discussions on timely topics. [A complete list appeared on p. 1614 of the December 1957 issue.] Two group luncheons were held, with Secretary Jordan and President Merryfield as featured speakers.

The following officers were elected: chairman—Quintin B. Graves, Professor, Dept. of Civ. Eng., Oklahoma A&M College, Stillwater; vice-chairman—Robert L. Lawrence, superintendent, Water Dept., Alexandria, La.; Arkansas trustee—R. L. Rimmer, general manager, El Dorado Water Corp., El Dorado; Louisiana trustee—P. G. Grambling, superintendent, Dept. of Water & Light, Ruston; Oklahoma trustee—J. K. Lindsey, consulting engineer, Tahlequah; Texas trustee—Henry J. Graeser, superintendent, Water Dept., Dallas. The following were appointed by the new officers to serve on the Board of Trustees: J. R. Pierce, chairman, Publication Committee; Fred Martin, chairman, Manufacturers' Committee; and Leslie A. Jackson, Secretary.

George H. West, water plant superintendent, Greater Lake Charles Water Co., Lake Charles, La., was nominated to receive the Fuller Award. Sara Elizabeth Tiner and Ben Behrens were granted the Egmont S. Smith Memorial Scholarship Fund awards. Sara Elizabeth is the daughter of J. B. Tiner, plant manager for Consolidated Chemical Industries, Inc., Bastrop, La., and Ben is the son of

(Continued on page 94 P&R)

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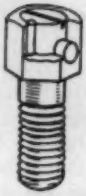

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Section Meetings

(Continued from page 92 P&R)

A. L. Beherns, maintenance superintendent, Water Dept., Eagle Lake, Tex.

LESLIE A. JACKSON
Secretary-Treasurer

West Virginia Section: The nineteenth meeting of the West Virginia Section was held at the McLure Hotel in Wheeling, Oct. 23–24, 1957. As has been the practice for the past 11 years, the meeting was held jointly with the West Virginia Sewage & Industrial Wastes Assn. The total registration for the meeting was 180 men and 10 ladies. Of this number, 115 men registered for the water works part of the meeting.

The technical sessions covered papers on settling basin design, coagulation, chlorine residuals, corrosion in action, coal washery wastes, and demineralization of water. A. E. Erickson, chief corrosion engineer of Peoples Natural Gas Co., presented the lecture on corrosion using a "mud tub," transparent ammeter and voltmeter, and various kinds of pipe. This demonstration was one of the high points of the meeting, in showing visually the action of corrosion and means of correction. [A list of technical papers and their authors appeared on p. 1615 of the December 1957 issue.]

A joint luncheon was held, at which Kenneth S. Watson, FSIWA president, was the guest of honor. Kenneth is a West Virginia boy and actively participated in Section affairs during the years he spent in the state. At the evening banquet, Thomas J. Blair Jr. was doubly honored, receiving both an AWWA Life Membership certificate and the nomination for the Fuller Award. Mr. Blair has been active in the water works business in West Virginia for many years and richly deserved the nomination. Wendell R. LaDue represented AWWA and contributed much to the success of the meeting and banquet. The Weirton Steel Men's Chorus entertained at the banquet. This, without doubt, is one of the outstanding men's choral groups in the United States.

The Section officers for the next year are: chairman—Wallace Grant, Charleston; vice-chairman—John B. Douglass, Charleston; trustees—H. K. Gidley, Charleston; Edward Shroyer, Wheeling; and Jordan Hugart, St. Albans; director—Dennis Clark, Bluefield; secretary-treasurer—Hugh W. Hetzer, Charleston; and assistant secretary—John Millar, Charleston. It was necessary to elect an additional trustee after receiving, with regret, notice that T. A. Stout was retiring because of ill health.

The number present and the interest shown is an indication of growth and increased activity of the West Virginia Section.

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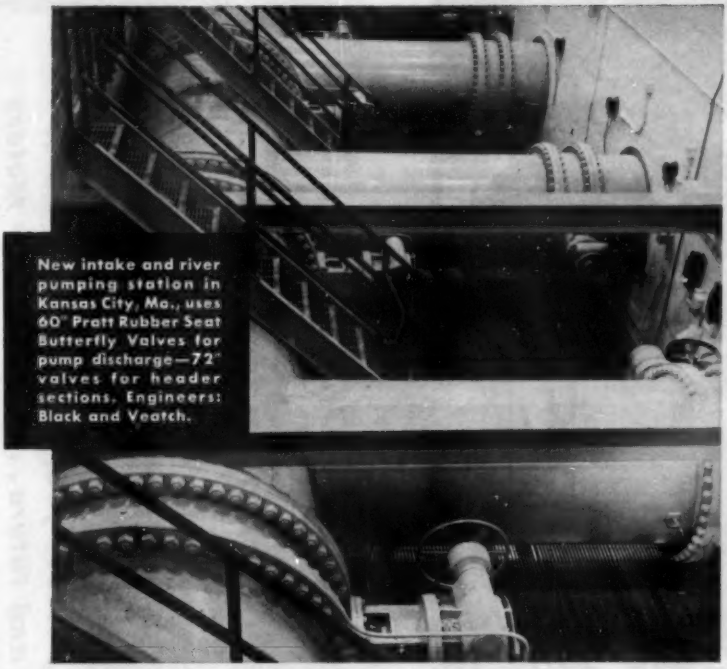

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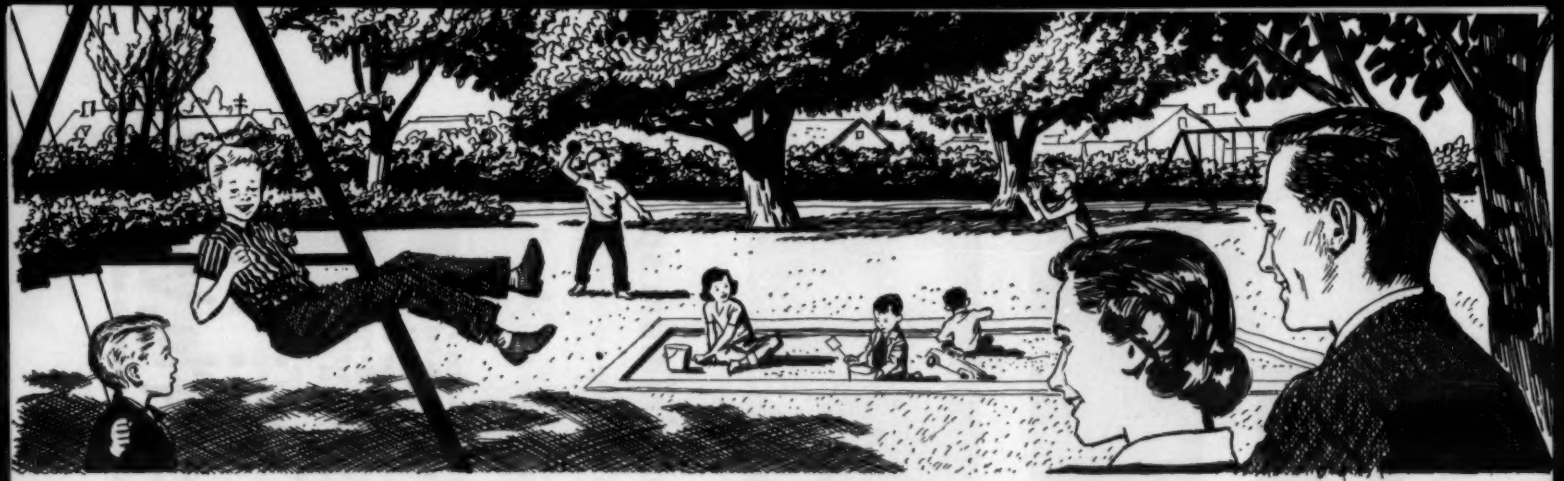

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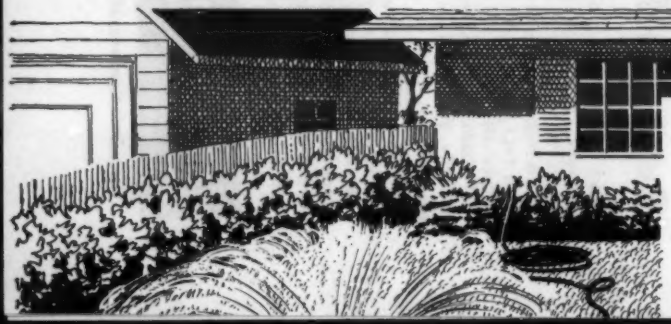

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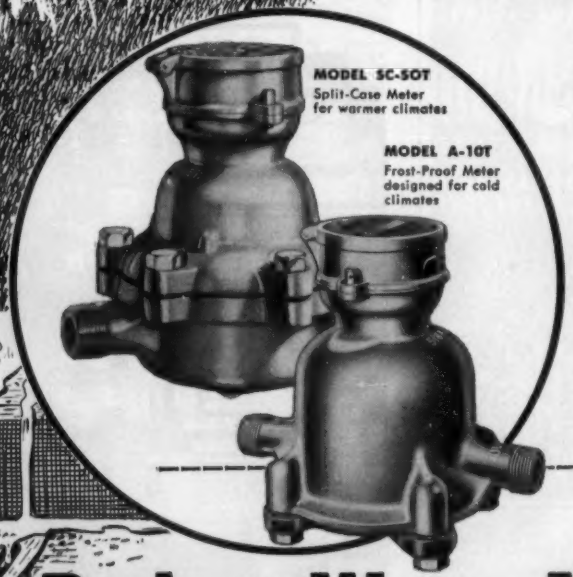

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NEW MEMBERS

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Alsing, Allen A., Water Supt., 101—1st St., Ashland, Ore. (Jan. '58) *D*

Arrighi, Victor M., Supervising Foreman, Water Div., Bd. of Public Works, Easthampton, Mass. (Jan. '58) *MRD*

Baier, Dwight C., Farm Advisor, Univ. of California Agricultural Extension Service, 1104 W. 8th St., Santa Ana, Calif. (Jan. '58) *R*

Baker, Jerome W., Supt., Water Meter Dept., Light, Gas & Water Div., Box 388, Memphis 1, Tenn. (Jan. '58) *M*

Baker, Wiley T., Assoc., W. D. Taulman & Assocs., Box 12286, Atlanta 5, Ga. (Jan. '58) *P*

Barrows, H. Lyndon, Jones Chems., Inc., 19200 N.W. 10th Ct., North Miami, Fla. (Oct. '57)

Brick, Del U., Engr., Water Dept., Great Falls, Mont. (Jan. '58) *MPD*

Bronkie, Robert T.; *see* Senior, Bissell & Bronkie

Burdick, Arthur C., Owner, Culligan Soft Water Service, 1278 E. Morgan Ave., Evansville, Ind. (Jan. '58)

Burris, George; *see* Greendale (Ind.) Utility

Carter, Ivan L., Jr., Borough Mgr., 53 W. South St., Carlisle, Pa. (Jan. '58) *MRPD*

Cherry, Irvine Chase, Asst. Chemist, Light, Gas & Water Div., 189 Madison Ave., Memphis, Tenn. (Jan. '58) *PD*

Colyer, John M., Engr., Smith & Gillespie, Box 1048, Jacksonville, Fla. (Jan. '58) *RPD*

Cooke, Howard W.; *see* Orange County (Calif.) Water Dist.

Covington, Robert Melvin, Supervisor, Water Meter Dept., Light, Gas & Water Div., Box 388, Memphis 1, Tenn. (Jan. '58) *M*

Dick, Richard I., San. Eng. Graduate Student, Univ. of Iowa, Iowa City, Iowa (Jr. M. Jan. '58)

Dickson, M. D. H., Partner, T. H. Newton Eng., Ltd., 1962 Avenue Rd., Toronto, Ont. (Jan. '58)

Duff, Earl L., Owner, Diamond Water Conditioning Co., Rte. 7, Box 159, Evansville, Ind. (Jan. '58)

Duvall, Sam E., Sanfax Co., 723 Bennett St., Greenville, S.C. (Oct. '57)

Edelstein, Erwin, Engr., Water Dist. Div., Dept. of Waters & Sewers, 404 City Hall, Chicago, Ill. (Jan. '58) *M*

Edmands, Horace John, Owner, Edmands Eng., 303 Phoenix Bldg., Bay City, Mich. (Jan. '58) *PD*

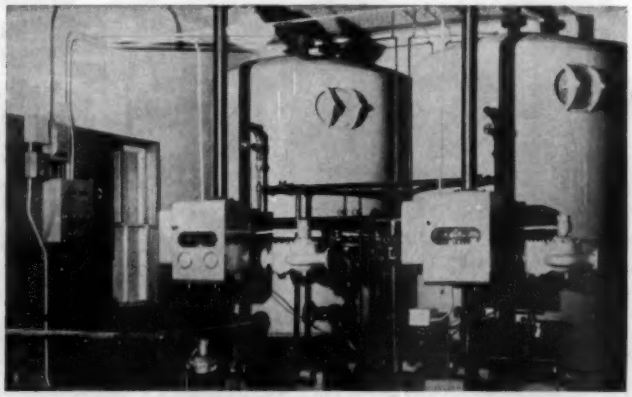
Elmore, John H., Dist. Supervisor, Munic. Waterworks, Box 286, 500 N. 3rd St., Burlington, Iowa (Jan. '58)

Englis, Duane T., Prof. of Chem., Univ. of Illinois, 321 Noyes Lab., Urbana, Ill. (Jan. '58) *RP*

Fahs, Bertram J., Public Utilities Records Dept., Remington Rand Div., Sperry Rand Corp., 315—4th Ave., New York, N.Y. (Jan. '58) *M*

Firth, William J., Field Engr., Infilco, Inc., Box 774, Roanoke, Va. (Oct. '57)

(*Continued on page 100 P&R*)



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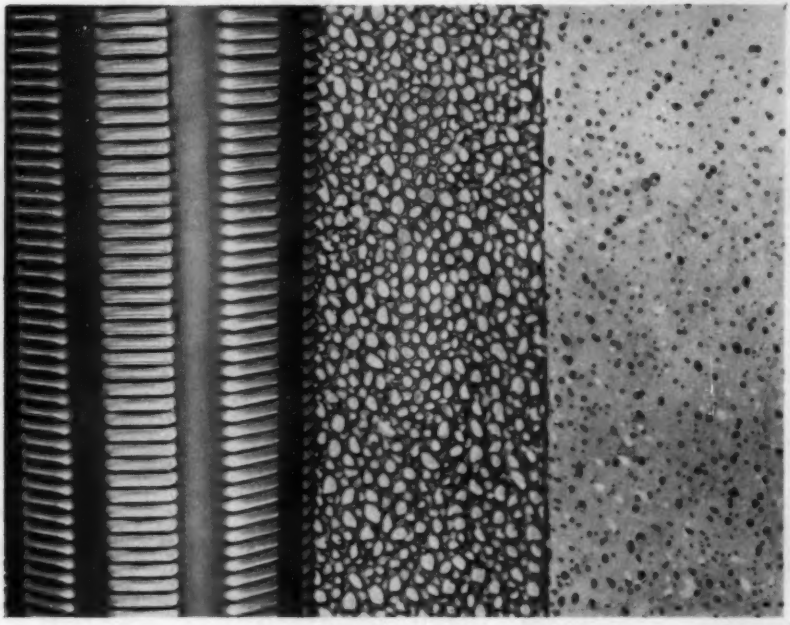

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(*Continued from page 98 P&R*)

Frazer, William W., Gen. Foreman, The Terre Haute Water Works Corp., 119 S. 7th St., Terre Haute, Ind. (Jan. '58) *D*

Freestone, Frank R., Supt., Ocean City Water Service Co., 10th & West Ave., Ocean City, N.J. (Jan. '58) *MPD*

Gallagher, Robert E., Asst. Supt., Water Distr. Dept., Light, Gas & Water Div., Box 388, Memphis, Tenn. (Jan. '58) *D*

Greendale Utility, George Burris, Mgr., 510 Ridge Ave., Lawrenceburg, Ind. (Corp. M. Jan. '58) *MD*

Grunwell, Harry, Asst. Engr., C. G. Russell Armstrong, 605 Bartlet Bldg., London St., W., Windsor, Ont. (Jan. '58)

Haltom City, City of, Sam Lucario, Director of Public Works, 5024 Broadway, Fort Worth 11, Tex. (Corp. M. Jan. '58) *M*

Hansen, Erling, Mgr., Waterworks, 1444 Main St., Marinette, Wis. (Jan. '58)

Harmon, David Burl, Asst. City Engr., 837 E. Jefferson St., Phoenix, Ariz. (Jan. '58) *MD*

Harris, Jay E., Gen. Contractor, 23 Parkway Ave., Crestview Hills, Covington, Ky. (Jan. '58) *D*

Hauser, Dale R., Contracting Engr., Pittsburgh-Des Moines Steel Co., 679 First National Bank Bldg., Chicago 3, Ill. (Jan. '58)

Hays, Samuel C., Pres., Emmitsburg Water Co., Emmitsburg, Md. (Jan. '58)

Holton, Philip J., Jr., Chief Engr., Water Supply Bd., Providence, R.I. (Jan. '58) *MRPD*

Hutchings, William Gates, Sales Repr., Johns-Manville Sales Corp., 777 Thomas, Seattle, Wash. (Jan. '58) *D*

Landolina, John A., Chief Engr., Pumping Stations, Light, Gas & Water Div., Memphis, Tenn. (Jan. '58) *MR*

Lange, Arnold J., Plant Engr., Bd. of Water & Light, Cedar St., Lansing 3, Mich. (Jan. '58) *MRD*

Leuty, Elbert J., Design Engr., Jones, Henry & Williams, 821 Security Bldg., Toledo 4, Ohio (Jan. '58) *PD*

Lintz, Clarence F., Asst. City Engr., Div. of Eng., 837 E. Jefferson, Phoenix, Ariz. (Jan. '58) *RD*

Lucario, Sam; *see* City of Haltom City (Tex.)

Lucas, City of, John N. Pinnick, Supt., Light & Water, Lucas, Kan. (Munic. Sv. Sub. Jan. '58) *MD*

Mahoney, Chester I., Supt., Water Dept., Box 327, Spiceland, Ind. (Oct. 57)

Marsell, Ray Everett, Prof. of Geology, Univ. of Utah, Salt Lake City 12, Utah (Jan. '58) *R*

Mathews, C. M., Mgr., Waterworks, Greenwood, Miss. (Jan. '58)

McCoy, Ted, Mayor, Brazil, Ind. (Jan. '58)

Montgomery, Millard R., Mgr., Culligan Soft Water Services, Inc., Box 57, Franklin, Ind. (Jan. '58)

Neave, John Wallace, Asst. Engr., Hazen & Sawyer, 122 E. 42nd St., New York 17, N.Y. (Jan. '58) *RP*

Norseth, Palmer, Eng., Bureau of Water, 209 City Hall, Portland, Ore. (Jan. '58) *MD*

Orange County Water Dist., Howard W. Cooke, Secy.-Mgr., 941 E. 1st St., Santa Ana, Calif. (Munic. Sv. Sub. Jan. '58) *MRPD*

Oswald, William Joseph, Asst. Prof. of San. Eng., Univ. of California, Berkeley, Calif. (Jan. '58) *RP*

Packard, Vernon Loring, Director of Public Utilities, Niagara Falls, N.Y. (Jan. '58) *MPD*

Parker, Samuel F., Sales Repr., Mueller Co., Decatur, Ill. (Jan. '58) *D*

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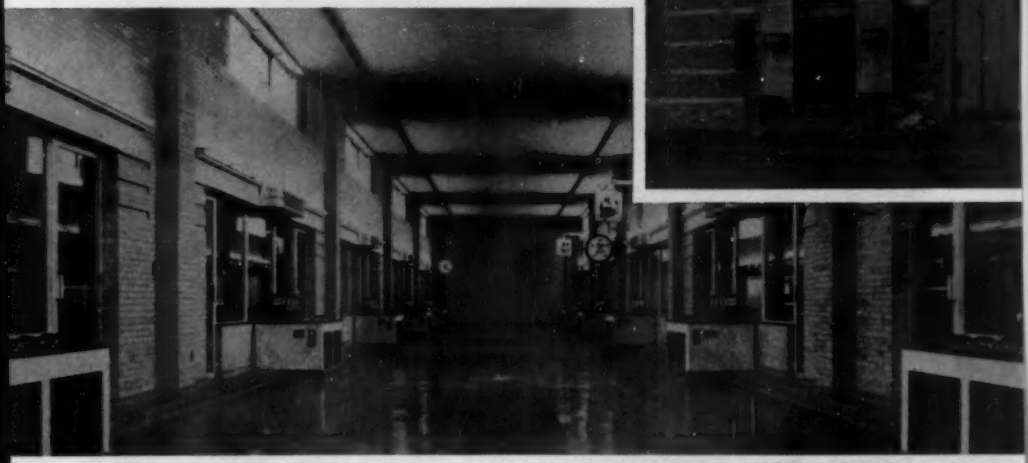

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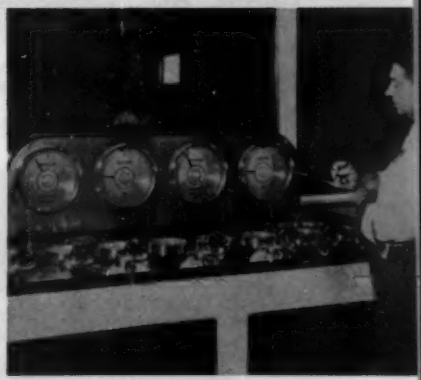

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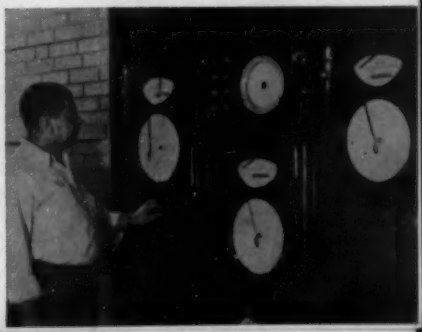


(*Continued from page 100 P&R*)

Person, Hjalmar Edwin, Asst. Engr. of Design, Waterworks, 2015 Commerce St., Dallas, Tex. (Jan. '58) *M*

Pinnick, John N.; *see* City of Lucas (Kan.)

Pool, William, Supt., Water Dept., Fort Branch, Ind. (Oct. '56)

Quisenberry, Ernest L., Town Mgr., Farmville, Va. (Jan. '58)

Ramstedt, Eugene Clayton, Asst. Civ. Engr., State Dept. of Water Resources, 1100 S. Grand St., Los Angeles 15, Calif. (Jan. '58) *R*

Readon, Robert; *see* Scottsburg (Ind.) Dept. of Waterworks

Reid, Frank, Jr., Assoc. Editor, Water Works Eng., 305 E. 45th St., New York, N.Y. (Jan. '58) *MRPD*

Robbins, Norman L., Sales Engr., Valve & Primer Corp., 341 W. Superior, Chicago, Ill. (Jan. '58)

Robertson, Frank E., Supt., Water Distr. Dept., Light, Gas & Water Div., Box 388, Memphis, Tenn. (Jan. '58) *D*

Robertson, James, Asst. City Engr., 1404 County-City Bldg., Seattle 4, Wash. (Jan. '58) *MR*

Rossetti, Anthony J., Pres., Rossetti Contracting Co., Inc., 4237 N. Lincoln Ave., Chicago 18, Ill. (Jan. '58) *MRP*

Rumley, Alkman J., Supt., Water Supply Dept., Light, Gas & Water Div., 488 N. Dunlap St., Memphis, Tenn. (Jan. '58) *R*

Schmerber, Louis J., Supt. of Sanitation, Gila County Health Dept., Globe, Ariz. (Jan. '58) *PD*

Scottsburg Dept. of Waterworks, Robert Readon, Supt., 500 S. Hyland St., Scottsburg, Ind. (Corp. M. Oct. '57)

Seeger, John C., Jr., San. Engr., Portland Cement Assn., 33 W. Grand Ave., Chicago 10, Ill. (Jan. '58) *PD*

Senior, Bissell & Bronkie, Robert T. Bronkie, Partner, 5757 Main St., Williamsville 21, N. Y. (Corp. M. Jan. '58) *RPD*

Smith, Garfield E., Chief Engr., Chester Valley Engrs., 159 W. Lancaster Ave., Paoli, Pa. (Jan. '58) *PD*

Stavrou, Constantine, Asst. Supt., Water Dept., 300 Walton Ave., South Orange, N.J. (Jan. '58)

Stricklan, James B., Asst. Chief Engr., Pumping Stations, Light, Gas & Water Div., Memphis, Tenn. (Jan. '58) *MR*

Stunkard, Joseph, Chief Engr., Water Works, Brazil, Ind. (Jan. '58)

Talbot, William F., Supt., Water Dept., 305 E. Monroe, Williamsport, Ind. (Oct. '57)

Tucker, Sam, City Engr., 837 E. Jefferson St., Phoenix, Ariz. (Jan. '58) *MRD*

Tutwiler, Thomas H., Distr. Engr., Light, Gas & Water Div., 179 Madison Ave., Memphis, Tenn. (Jan. '58) *D*

Walsh, John H., Gen. Foreman, Water Dept., Wichita, Kan. (Jan. '58) *D*

Weeber, Earl R., Cons. Engr., Hamilton, Weeber & Ward, 356 Houseman Bldg., Grand Rapids, Mich. (Jan. '58) *RPD*

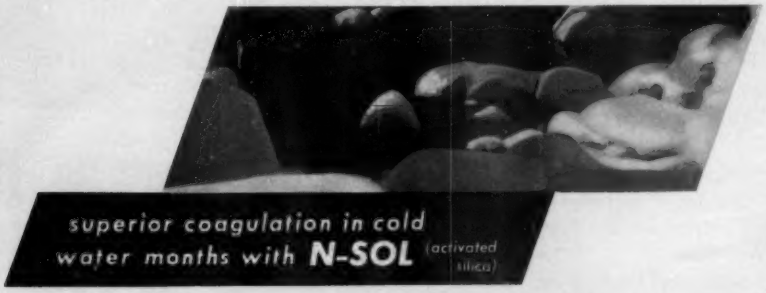
Wild, Albert Earl, Mgr., Idaho Water Co., 114 S. 4th St., Couer d'Alene, Idaho (Jan. '58) *M*

Williamson, Edward Kenneth, Director, Public Works, El Dorado, Ark. (Jan. '58) *RD*

Winkel, Adrian P., Comr. of Public Works, Bd. of Water Comrs., Saint Paul, Minn. (Jan. '58)

Yelser, Ted M., Mgr., Plastic Pipe Tech. Services, Crescent Plastics, Inc., 955 Diamond Ave., Evansville 7, Ind. (Jan. '58) *D*

Zerfas, Howard R., Culligan Soft Water Service, 205 N. Commerce St., Portland, Ind. (Jan. '58) *P*



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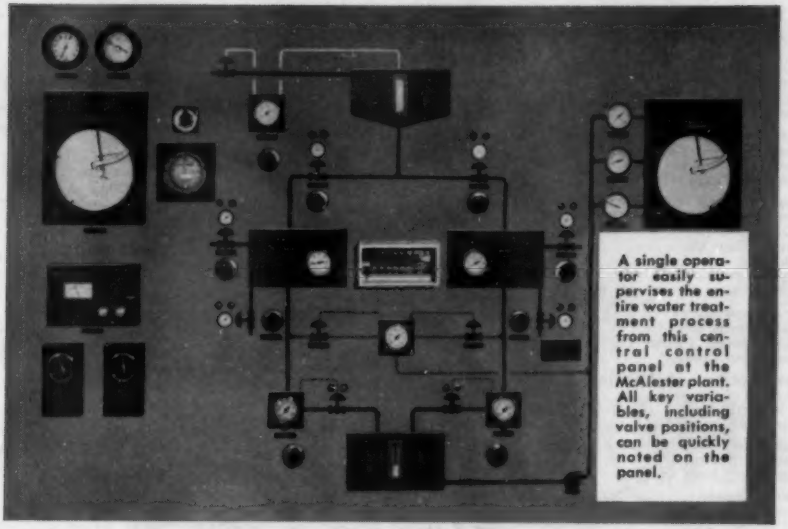

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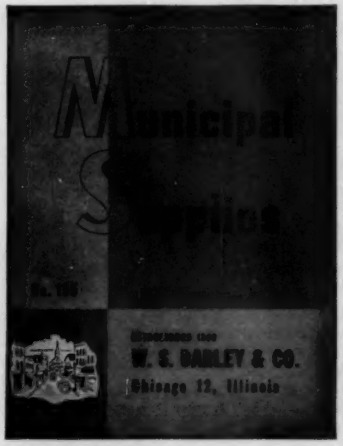

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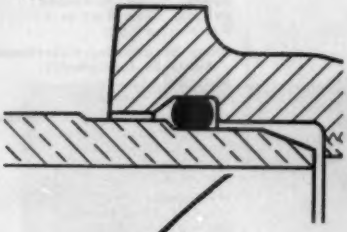

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General Filter Co.
Infilco Inc.
Omega Machine Co. (Div., B-I-F Industries, Inc.)
Permutit Co.
Wallace & Tiernan Inc.

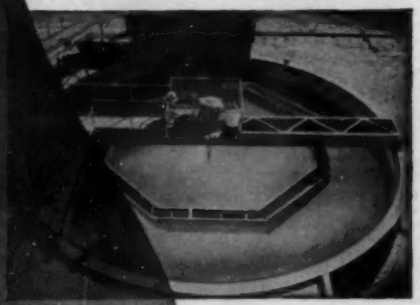
Magnetic Dipping Needles:
W. S. Darley & Co.

Meter Boxes:
Ford Meter Box Co.
Pittsburgh Equitable Meter Div.

Meter Couplings and Yokes:
Badger Meter Mfg. Co.
Dresser Mfg. Div.

Why General Filter?

There are many reasons why municipalities and industries have installed General Filter water treating plants:



INSTALLATION SUPERVISION . . . General Filter "job-engineers" each installation to the consulting engineer's specification . . . supervises the installation and trains the personnel who will work with the equipment.

OPERATIONAL DEPENDABILITY . . . for twenty years General Filter has concentrated all of its efforts toward "better water treatment". Their efforts have produced water treating plants and equipment that are *completely* dependable.

ENGINEERING KNOW-HOW . . . General Filter's design and construction engineers are familiar with the problems involved in water treatment.

GREATER ECONOMY . . . the only real test of economy is [illegible] test. General Filter plants stand up over the years providing "better water" with minimum maintenance, longer trouble-free, smoother operation.

Find out why you should specify General Filter . . . Write today for detailed information regarding your water treatment problems.

General Filter Company
AMES, IOWA
. . . *better water*

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REVENOUS ZEOLITE • IRON RUST REMOVAL • DEMINERALIZATION • SOFTENERS

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Meter Reading and Record Books:
Badger Meter Mfg. Co.

Meter Testers:
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Ford Meter Box Co.
Hersey Mfg. Co.
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Pittsburgh Equitable Meter Div.

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Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
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Burgess-Manning Co., Penn Instruments Div.
Hersey Mfg. Co.
Neptune Meter Co.
Pittsburgh Equitable Meter Div.
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American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
James B. Clow & Sons
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Cement Lined:
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Cast Iron Pipe Research Assn.
James B. Clow & Sons
United States Pipe & Foundry Co.
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Layne & Bowler Pump Co.
Worthington Corp.

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Layne & Bowler Pump Co.
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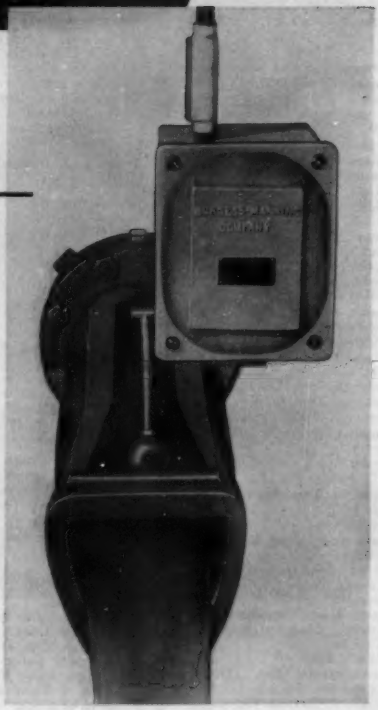

The BURGESS-MANNING Type "MN" Meter

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Stilling well and cover
Connecting pressure piping and fittings
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Clear water for flushing
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Here is a distinct advance in the art of measuring flow of liquid under gravity conditions. Here is accurate measurement of flow at all times. Here is a method of measurement that is free of troublesome and highly objectionable maintenance.

Think of the savings! Think of the elimination of maintenance!

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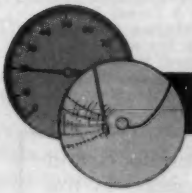

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Zeolite: see Ion Exchange Materials

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It takes just two steps to assemble the FLUID-TITE Coupling. Lubricate the tapered edge of the gasket. Then slide in the pipe.

Major Advance in Waterworks Industry. K&M's exclusive FLUID-TITE Coupling provides permanent, water-tight, root-tight connections.

SLIDE IT IN QUICK... IN 2 EASY STEPS

IT'S "K&M" ASBESTOS-CEMENT PRESSURE PIPE WITH EASY-TO-INSTALL FLUID-TITE COUPLING

Installation is fast and economical! It doesn't require skilled labor, heavy machinery, or heavyweight coupling pullers. Install it in any weather.

The seal grows tighter as the pressure climbs! Coupling rings expand as water mains fill. Rings have holes on one side for self-energizing action.

It's practically indestructible! "K&M" Asbestos-Cement Pressure Pipe is non-tuberculating, non-electrolytic, and corrosion-resistant. Its first cost is often the last cost. Pressure remains normal—pumping costs stay low.

Write today for more information.

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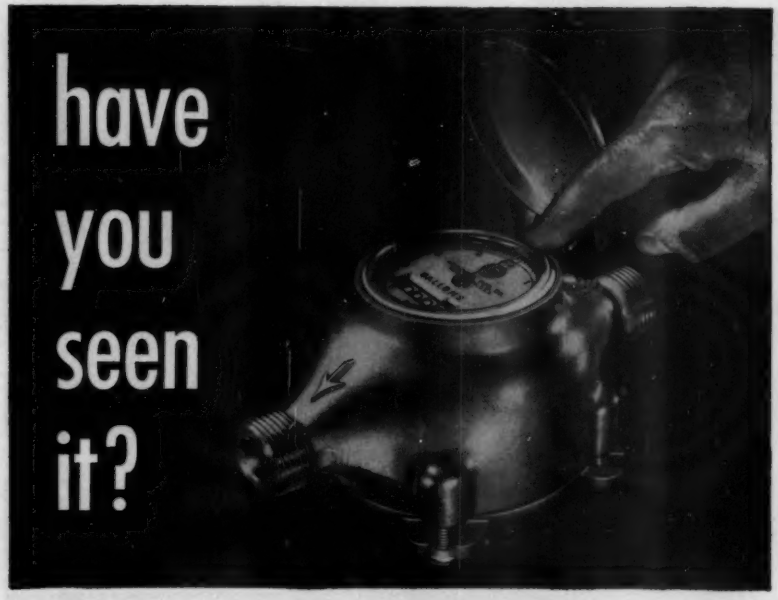


The World's First Sealed Register Water Meter

Hermetically Sealed Register
Never Fogs—Always Easy To Read

All the register parts and reduction gearing are hermetically sealed in this stainless steel housing having a thick, heat treated glass window. Condensation, dust and corrosion are permanently excluded. A powerful magnetic drive eliminates the need for a stuffing box.

Everything about the Rockwell Sealed Register Water Meter makes history. Makes sense, too. It's the first water meter to operate successfully without a troublesome stuffing box. A simple, powerful magnetic coupling does this work—without friction, binds or leaks. The hermetically sealed register forever eliminates condensation problems that often make ordinary meter registers hard to read. Ends corrosion problems, too.

Even Rockwell has never made a more accurate meter. It requires less maintenance than any meter you have ever used. And it's tamperproof. The cost? No more than an ordinary meter—actually less than meters now fitted with special types of registers. Write for bulletin. Rockwell Manufacturing Company, Pittsburgh 8, Pa.

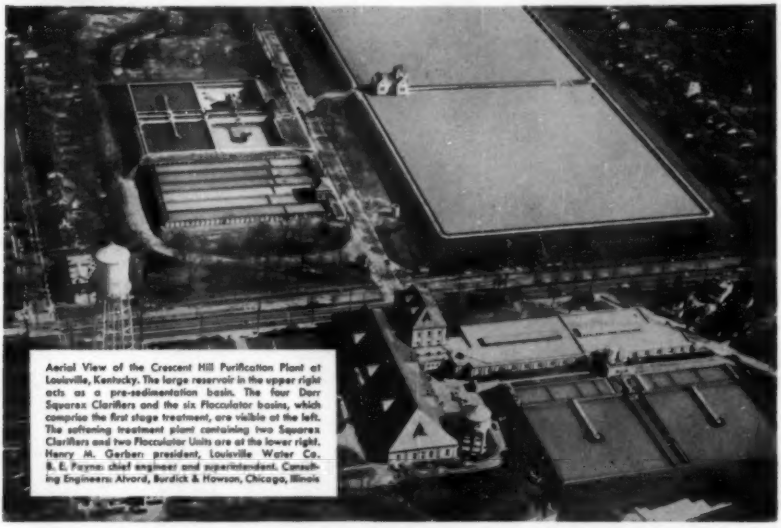
Rockwell Sealed Register Water Meter

With Powerful Magnetic Drive

The solution to this problem is always the same . . . but

Water Treatment Problems are different

No two water treatment problems are exactly alike. The right solution to each can only be arrived at after a careful study of the local conditions. Variables such as raw water composition, rate of flow and results required automatically rule out the cure-all approach. The installation shown below is a good example of how equipment should be selected to fit the job . . . and not vice versa.



Aerial View of the Crescent Hill Purification Plant at Louisville, Kentucky. The large reservoir in the upper right acts as a pre-sedimentation basin. The four Dorr Squarex Clarifiers and the six Flocculator basins, which comprise the first stage treatment, are visible at the left. The softening treatment plant containing two Squarex Clarifiers and two Flocculator Units are at the lower right. Henry M. Gerber: president, Louisville Water Co. B. E. Payne: chief engineer and superintendent. Consulting Engineers: Alvord, Burdick & Howson, Chicago, Illinois

Louisville, Kentucky

Adds More Dorr Squarex Clarifiers for Increased Capacity

One hundred years ago the Louisville Water Company began construction of their water plant facilities with the erection of their first pumping station. In the early thirties six Dorrco Flocculator Units were installed for coagulation with a design capacity of 60 MGD. In 1945 two type SZ Dorr Squarex Clarifiers, each 200′ square, and two Flocculator Units for basins 176′ long, were installed inaugurating Louisville's softening treatment plant.

By 1954 rapid population increases necessitated a proportionate increase in water supply requirements, and four Dorr Squarex Clarifiers, each 150′ square, were installed in the first stage increasing the design capacity to 160 MGD.

Two additional Dorr Squarex Clarifiers, each 200′ square, and two type P Flocculator Units, for basins 179′ long and 50′ wide, have been ordered, which will provide softening for 160 MGD.

If you would like more information on the complete line of Dorr-Oliver Water Treatment Equipment, write Dorr-Oliver Incorporated, Stamford, Conn., for a copy of Bulletin No. 9141.

Every day over 8½ billion gallons of water are treated by Dorr-Oliver equipment

Squarex and Flocculator trademarks reg. U. S. Pat. Off.

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WORLD-WIDE RESEARCH • ENGINEERING • EQUIPMENT
STAMFORD • CONNECTICUT • U. S. A.

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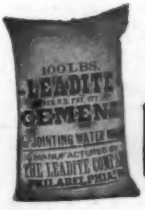
Jointed for . . . Permanence with LEADITE

Generally speaking, most Water Mains are buried beneath the Earth's surface, to be forgotten,—they are to a large extent, laid for permanency. Not only must the pipe itself be dependable and long lived,—but the joints also must be tight, flexible, and long lived,—else leaky joints are apt to cause the great expense of digging up well-paved streets, beautiful parks and estates, etc.

Thus the "jointing material" used for bell and spigot Water Mains MUST BE GOOD,—MUST BE DEPENDABLE,—and that is just why so many Engineers, Water Works Men and Contractors aim to PLAY ABSOLUTELY SAFE, by specifying and using LEADITE.

Time has proven that LEADITE not only makes a tight durable joint,—but that it improves with age.

The pioneer self-caulking material for c. i. pipe.
Tested and used for over 40 years.
Saves at least 75%



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Girard Trust Co. Bldg. Philadelphia, Pa.